3rd International Symposium on

Connections between Steel and Concrete

Stuttgart, Germany

27th - 29th September 2017

Proceedings

Organised by
Institute of Construction Materials,
University of Stuttgart

Co-sponsored by
ACI, fib, DAfStb, ECS

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Akanshu Sharma
Jan Hofmann
Preface

Steel is an integral part of any concrete construction either in the form of anchors connecting structural/non-structural components to the structure or in the form of reinforcement embedded in concrete or sometimes as a part of steel-concrete composite construction. To ensure structural integrity, it is essential to form a reliable connection between steel and concrete. The anchorage must serve its function well to ensure the interaction between the components connected to the structure and the structure itself. The bond between reinforcement and concrete must be ensured to maintain the required flow of forces from concrete to steel and vice-versa. The concrete and steel structural members must be able to interact with each other in a desired way. If the connection between steel and concrete loses its integrity, the integrity of the entire structure comes in jeopardy. In order to ensure the integrity of connections between steel and concrete, it is required to (i) investigate their behavior through high quality experiments, (ii) evaluate their performance with advanced and reliable computational methods, and (iii) summarize them in practical and reliable design rules and models to be used by engineers and practitioners.

Although connections between steel and concrete are used ever since the commencement of reinforced concrete construction, there are several aspects which are yet to be fully understood. Advancement in technologies lead to development of new products such as post-installed anchors, anchor channels, high strength reinforcing bars etc., whose performance in concrete need to be investigated. New concrete based materials such as high performance concrete, fiber reinforced concrete, geopolymer concrete etc. are being developed and their compatibility with the steel components and rebar must be verified. The interaction between structure and the anchorages or concrete and steel structure connected through anchorages need to be understood. Innovative fastening solutions between steel and concrete components are being developed for which reliable design models are needed. Furthermore, several general design issues, e.g. anchorages with supplementary reinforcement, fatigue behavior of anchors, long term performance of bonded anchors, anchor channels under different loading combinations, harmonization of design between post-installed and cast-in place reinforcing bars, influence of corrosion on performance of anchorages and reinforcement etc. need in-depth investigations.

In addition, globally, structures are exposed to natural and man-made hazards more than ever, which induce extreme loads on the structural components, e.g. earthquakes, fire, impact and combinations thereof. The performance of existing structures against such hazards has exposed several inherent weaknesses in the past. Often, the performance of structures under such hazards is significantly influenced by the performance of connections between steel and concrete. Since the loads induced in the structures by such hazards cannot be reliably estimated and also since under such loads, compatibility requirements between different components result in additional demands on different components, the designs can only be reliably done following a performance based design approach. Similar approaches must also be developed for the design of connections between steel and concrete to ensure a reliable performance from the connections and in turn from the structures against extreme hazards.
One of the most important applications of anchorages is in strengthening of structures for connecting the strengthening element and the parent structure. Every strengthening needs a certain type of anchorage and the performance of the strengthening depends largely on the performance of the anchorage itself. The demands posed on the anchorages used in strengthening (e.g. seismic strengthening) could be very challenging, which includes high forces, large crack widths, combined load and crack cycling, limited area and depth to develop the required resistance etc. In order to ensure the safety of structures, therefore, it is essential to develop innovative strengthening methods along with anchorage techniques that would allow the strengthening to serve its desired function.

The 3rd International Symposium on Connections between Steel and Concrete (ConSC2017) aims at bringing the experts from around the globe working in the field of steel-concrete connections to share and discuss the current state-of-the-art as well as the possible future directions to improve the safety of structures by making reliable connections between steel and concrete. The previous two symposia were held in Stuttgart in 2001 and 2007. This symposium offers a platform for the academicians, researchers, industry, regulatory authorities, practicing engineers and students to share the knowledge gained through a decade of research and development. With more than 120 high quality research papers along with the keynote addresses and invited lectures from the renowned experts, this symposium is a significant step forward towards the enhancement of knowledge as well as to provide food for thought for the forthcoming research in the field of connections between steel and concrete.

Akanshu Sharma

Jan Hofmann
Institute of Construction Materials –
more than fastening and strengthening methods

The Institute of Construction Materials (IWB) has a long history within the University of Stuttgart and the Material Testing Institute (MPA, University Stuttgart). The Material Testing Institute was founded in 1884 at the “Polytechnikum Stuttgart” by Carl von Bach and later headed by Otto Graf until 1950. In 1980, the Otto-Graf-Institute was integrated to the Ministry for Economics of Baden-Württemberg by Gallus Rehm. With this change, one department of the Otto-Graf Institute remained at the faculty of Civil Engineering, University of Stuttgart. This department was named the “Institute for Construction Materials” and the name is retained for the last 37 years and beyond.

Nevertheless, many things changed during this time. In 1984, a new professorship for fastening technology was established at the IWB and Rolf Eligehausen became the Professor and the head of the department for fastening technology. This was the start of a new research field in addition to the field of construction materials and bond between reinforcement and concrete. The field of fastening technology developed very fast and brought ahead the research for different fastening systems such as headed studs, expansion anchors, bonded anchors or concrete screws. Starting with 1997, Rolf Eligehausen set up a laboratory for the department of fastening technology that was specialized mainly in fastenings in concrete. With this time, the IWB became an important center for fastening technology in the world and the knowledge transfer to standards and guidelines came into the focus.

However, it is not only an honor to continue this success and knowledge but also a mission to improve and diversify to other fields that belong directly or indirectly to fastening technology. In 2009, the department was taken over by Jan Hofmann and since this time new research fields were established in the department like the “strengthening methods” and the “gluing techniques”. Furthermore, the research fields for fastenings technology were strongly enlarged and diversified.

Today, the department of fastening and strengthening methods of the IWB deals with fastenings in concrete, masonry, natural and artificial stones as well as special materials. Today not only concrete and masonry is our expertise but everything where fixings are used. Also the scope of research changed in a way that the test results and materials must be assessed with respect to time dependent performance or performance against hazards. For fastening technology, this is the most important research area currently. Therefore, the focus changed from static or quasi-static and short term testing to dynamic and long term testing including simulating hazard effects like fire or impact. In addition, the time dependent effects like permanent or fatigue loads or special environmental impacts like corrosion are getting more and more important in most applications. During the past 10 years, the field of research and expertise at IWB has therefore strongly increased. Today we are 4 professors (Jan Hofmann, Akanshu Sharma, Werner Fuchs and Josko Ozbolt) at the department of fastening and strengthening methods. We are experts for testing and simulating applications for fastening and strengthening methods. Especially the possibility to simulate the mechanical (damage, fatigue, impact …) and physical processes (corrosion, fire …) in concrete in a time dependent way is our unique selling point. We can combine testing and simulations to get the best results and to understand the basic mechanisms for

- fastenings in concrete, masonry, natural stones, glass and other materials
• fastenings under arbitrary loading regimes (fatigue, sustained loading) and environmental impacts (UV radiation, CO2, SO2, freeze thaw, …) and corrosion
• strengthening of concrete structures for hazard impacts (earthquakes, impacts, fire, …)
• bond behavior of cast in and post installed reinforcing bars using the beam end test for static and cyclic loading and after hazard impacts.
• direct fasteners and glued fasteners or glued systems for concrete and masonry surfaces without drilling

This expertise we keep with responsibility but also with passion because also we want to learn new things every day. That is our philosophy. So we support you with research, testing and knowledge transfer for your products, systems and innovations and provide solutions that are safe, reliable and practical.
Symposia on ‘Connections between Steel and Concrete’ -

a brief history

Anchorage by post-installed fasteners and cast-in anchors has seen dramatic progress in research, technology and application between 1980 and 2000. The understanding of the fundamental principles underlying both disciplines had significantly improved. There has been also rapid growth in the development of new products on the basis of actual research, as well as the establishment of first international directives and codes to ensure their safe and economic use in a wide range of structural applications. A similar process could be observed in the field of composite structures. However, both disciplines were driven by separate developments:

Whereas the research in composite construction was more focused on the global behavior of the structures, in fastening technology the local behavior of the fastening system was the priority. Additionally, the advancement in material science led to development of high strength reinforcing bars as well as new concrete based anchorage materials. It became apparent that a forum offering the opportunity to expand and to exchange experience in the field of connections between steel and concrete would benefit all involved. Furthermore this forum should aid in the rapid dissemination of new ideas, technologies and solutions. New areas of research should also be detected.

Therefore the first Symposium on Connections between Steel and Concrete was held in Stuttgart from September 10th to 12th, 2001. More than 400 experts from all facets of the research, design, construction and anchor manufacturing community from around the world attended the symposium. More than 120 presentations were made covering the topics of testing, behavior and design, durability, exceptional applications, strengthening and structures in the fields of anchorage to concrete and composite structures as well as related topics such as anchorage to masonry.

Six years later, after outstanding research efforts and progress in both the disciplines, fastening technology and composite construction, resulting in fundamental principles and innovative solutions increasingly employed in the design, execution and retrofit of engineered structures, and motivated by the success of the first Symposium on ‘Connections between Steel and Concrete’ the second Symposium was held between September 4th and 7th, 2007. Again more than 120 presentations were performed and the symposium was attended by around 400 delegates. They took the opportunity to meet and interact with colleagues which allowed fruitful discussions and exchange of knowledge.

The primary objective of the previous Symposia on ‘Connections between Steel and Concrete’, that is to provide a forum for the presentation and discussion of the state of the art as well as new research developments in the field of design concepts for connecting the two materials was successfully achieved.

In the meantime, 10 years were spent in extensive research and work in code committees yielding international harmonization in design and testing procedures. Therefore ConSC 2017 is organized to follow the tradition of the previous Symposia and provide the base for excellent exchange of knowledge for all professionals active in the field of connections between steel and concrete.
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INVITED LECTURES

SPECIAL SESSION: 75TH BIRTH ANNIVERSARY OF PROF. ROLF ELIGEHAUSEN

Role of fiber reinforcement on lap splices behavior
G. Plizzari, G. Metelli

Assessment of fasteners to concrete - a tribute to Rolf Eligehausen
L. Elfgren, R. Nilforoush, M. Nilsson, U. Ohlsson

Numerical Simulation of Fastenings - Reminiscenes from IWB
V. Cervenka

PLENARY LECTURE

Anchorages in concrete construction: past, present and future
R. Elighausen
KEY NOTE LECTURES
POTENTIALS IN CONNECTIONS BETWEEN STEEL AND CONCRETE
WITH SPECIAL EMPHASIS ON TEMPERATURE INFLUENCES

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ABSTRACT

Modern construction technology requires new types of constructions materials and new construction elements both for existing as well as for new structures. Adequate interaction between elements and bond between reinforcement and concrete are major issues for concrete and for steel-concrete composite structures. High requirements have to be fulfilled both for fastening and connecting elements as well as for the bond interaction of reinforcements on anchoring (force transfer) capacity, ductility, deformation capacity, safety, long term properties, durability, fire resistance, applicability etc.

Bond research is again in the focus of interest owing to the appearance of new types of reinforcements (e.g. non-metallic reinforcements) and new types of concretes. New materials provide new aspects to the anyhow complex picture of influencing factors and failure modes. Bond performance has effects both on ultimate behaviour (flexure and shear capacities) and serviceability behaviour (cracking, tension stiffening and deflections).

In some areas of bond and interactional behaviour (especially high temperatures) still limited information is available. The reason is the complexity of experiments. Therefore, a particular emphasis is given herein to consequences of high temperatures.

Present paper intends to review some of the new potentials in anchoring solutions as well as for bond.

1 Introduction

The series of conferences on Connections between Steel and Concrete provides an excellent forum for presenting developments on connections of all kinds including (i) fastenings/anchors, (ii) bond in general and (iii) steel-concrete composite structures.

Eligehausen and Fuchs called our attention as current and future research projects for fastening technique in their Keynote at the 2nd ConSC20071: optimization of the design methods for fastenings, behaviour and design of fastenings with anchor reinforcement, fire resistance of fasteners, fastenings
under seismic excitations, strengthening and retrofitting of structures, new fastening techniques, influence of concrete composition on the behaviour of chemical anchors, durability of chemical anchors, connections with post-installed rebars, fastening in solid and hollow masonry.

On the other hand, the following developments were addressed for composite structures by the Keynote of Kuhlmann at the 2nd ConSC2007 as: composite girders and slim-floor girders, composite slabs, composite columns, composite joints and frames, composite bridges.

Present paper intends to give an overview on potentials and new developments of connections mainly to above (i) and to (ii) with special emphasis for temperature influences on them, respectively.

1.1 Interaction between anchors and concrete, behaviour and modelling

Anchoring methods and anchoring systems are continuously further developed. A new element is shown in Fig. 1 as a special example. This is short (about 300 mm long) GFRP bar, where the overall length is too short to develop enough anchoring capacities at both ends. Therefore, multi diameter heads were created with increasing diameters outwards to provide improved development of anchoring forces.

![Figure 1: New type of GFRP bar with end-anchors at both ends](image)

A significant part of research is dealing with the strengthening of reinforced concrete (RC) members with externally bonded fibre-reinforced polymer (FRP) composites has been generated to date e.g. 3, 4, 5, 6, 7. Researchers have consistently identified a limitation of the approach to be debonding of the FRP in generally a brittle manner. In addition, debonding has been found to typically occur at strains considerably lower than the strain capacity of the FRP material e.g. 7, 8. In a bid to better utilise the FRP material and provide a safer strengthening solution, anchorage devices may be installed. Devices such as FRP anchors, U-jackets and nailed plates have proven to be effective in suppressing debonding failures and they have enabled strengthened members to achieve greater deformability in cases 9, 10, 11.

Whenever the strengthening fibres are not pre-impregnated by resin, the anchoring capacity can be developed within a short distance by creating special end solutions like in Fig. 2.
There are several reasons to pursue research and application of FRP anchors in comparison to anchors made of more traditional materials such as metal, namely, (i) FRP anchors can be applied to wide shaped FRP-strengthened members such as slabs and walls, (ii) FRP anchors are corrosion resistant, (iii) the flexible nature of FRP anchors results in limited bearing pressure induced between the anchor and adjacent FRP plate which minimizes plate splitting failure, and (iv) the FRP anchor can be installed at the same time as the FRP plate thus ensuring a monolithic plate and anchor strengthening unit.

Several post-installed anchors (Figure 3) are available with different ways of force transfer mechanisms in order to enable force applications owing to technological reasons or any other reasons. The fastenings can transfer the load to the base material via the following mechanisms: mechanical interlock, friction or bond. Furthermore, the most recent techniques use combined bond and friction (e.g. bonded expansion anchors). In case of expansion anchors, the load is transferred by friction. Generally, an expansion sleeve is expanded by an exact displacement or torque. Chemical fastenings are anchored by bond. Bonded anchors can be divided into two subgroups: capsule or injection systems. The bond material can be either organic, inorganic or a mixture of them. In this

Figure 2: New ideas for anchoring FRP [to a): Ref. 12 and to b): Ref. 13]
case the loads are transferred from the steel (normally a threaded rod, rebar) into the bonding material and are anchored by bond between the bonding material and the sides of the drilled holes 14, 15, 16, 17, 18, 19, 20, 21, 22.

The load balancing for fastenings is visualized in Figure 3.a with or without the presence of a crack in concrete within the section of the anchor. Figure 3.b indicates how the force that is generated by the anchor in concrete balanced by the steel reinforcement. Figure 3.c gives the main types of failure modes in concrete. Figure 3.d indicates the complex behaviour of an anchor group close to a corner.

Load bearing of fastenings can be determined by taking the minimum of ultimate loads corresponding to different failure modes.

Steel failure depends on the tensile strength of the steel rod. Steel capacity can be calculated from the ultimate steel strength ($f_{u}$) and the cross-sectional area ($A_s$) 14, 15, 16.
The properties of concrete cone failure mostly depend on embedment depth ($h_{ef}$) and concrete strength ($f_{ck}$). Cone failure is the optimal failure type, because concrete strength is completely utilized. Partial cone failure is a common failure type of bonded anchors; in this case the bond between the bond material and concrete is partly damaged. This means a transitional failure type between cone failure and pull-out $^{14,16}$.

Pull-out failure has to be discussed separately for bonded and expansion anchors. Pull-out failure of mortar bonded anchors means bond failure between mortar and concrete, while pull-out failure excluding mortar means bond failure between the steel fastening and the bonding material. The bond strength ($\tau_{u,k}$) depends on the certain product, but its value is included in the corresponding approvals.

Pull-out failure in case of expansion anchors is possible under tension, including or excluding the expansion sleeve (pull-out/pull-through).

Splitting failure is caused by the critical edge-, spacing distances. Load bearing capacity can be influenced by distances from edges and by spacing distances; these effects can be taken into account by reduction factors $^{14,16}$.

The use of precast concrete elements in construction has increased over the last decades. It arises from the innovation of new construction methods and usually advances beyond standard rules of design. These individual elements must be transported to the building site and affixed in the corresponding place thereby connecting them with the main structure or with other auxiliary elements. Anchor elements previously incorporated in the precast pieces serve to perform this function. Every load that affects these elements is transferred through their anchorages and makes them an essential part of the structural support system. Thus, the proper design of these elements is essential $^{25}$.

If fasteners are subjected to repeated actions, the fatigue resistance must be verified. Since, the fatigue behaviour of steel is well known for fasteners, a special attention should be devoted to the fatigue of concrete. The concrete fatigue can be decisive in case of high cycle fatigue loading, particularly if the load amplitude is selected such, that it is below the endurance limit of steel. In case of fasteners, no endurance limit can be observed, which corresponds to the expected concrete fatigue behaviour $^{26}$.

The durability of the prestress anchor bolt is strongly affected by the time-dependent behaviour of concrete. A concrete with a mix proportion giving the lowest possible shrinkage and creep is recommended in order to limit the decrease in the prestress. Moreover, these anchors are submitted to fatigue stress which can increase the concrete packing $^{27}$.

Our experimental results on bonded anchors after fire exposure are presented in Ch 2.2.

After temperature loading the material characteristics could be significantly changed $^{28,29,30,31,32}$. In case of bonded anchors glass transition temperature of the adhesive is important. Bond strength is considerably reduced if the temperature goes beyond glass transition temperature of the adhesive.
Metallic post-installed and undercut anchors were experimentally studied by Bamonte, Gambarova \(^{31}\) in thermally damaged concrete. The shank diameter was 10 mm. The effective depth was 80 mm. The anchors were installed into the previous heated surface. The observed peak load was linearly decreasing by increase of the previous temperature load \(^{33, 34, 35}\). The failure mode is also affected by the temperature. At room temperature failure of the steel shank took place.

Ožbolt, Kožar, Eligehausen and Periskič \(^{36}\) indicated by FEM analysis that the largest reduction of the load bearing capacity is obtained for anchors with relatively small embedment depth. By heating of concrete the resistance is generally decreasing, however, when the concrete member is heated than cooled down, the resistance can increase and it can even be larger than the resistance of the anchor in unheated concrete \(^{36}\).

### 1.2 Bond between steel and concrete

Bond performance in reinforced concrete is usually represented by bond stress vs. slip (\(\tau_b - s\)) relationships evaluated from pull-out tests (Figure 4). In general, the following phases of bond performance can be distinguished: (1) adhesional contact without slip, (2) mechanical interlock with increasing slip (during mechanical interlock micro-cracks \(^{37}\) form around the reinforcement in the concrete and micro-crushing takes place in front of the ribs of reinforcement), (3) after reaching the maximum stress (bond strength) the small concrete teeth between ribs of the reinforcement are sheared off, (4) finally only friction is provided also for plain as well as for ribbed bars. This residual bond strength for plain bars is about two-third of that of ribbed bars owing to the differences in frictional coefficients for concrete to steel (in case of plain bars) or concrete to concrete (in case of deformed bars). Increase of bond stress plain bars after adhesion is very low and its value remains practically constant during pull-out.

Two types of failure modes are distinguished: (a) pull-out failure if adequate confinement is provided by the concrete cover or by transverse reinforcement of (b) splitting failure with drop of bond stress if concrete cover splits along the reinforcing bar. Eligehausen \(^{38}\) called the attention for the importance of splitting failure especially for lap splices but also for anchorages.

Bond research has a history of at least 100 years and never seems to stop. The reason is probably the very wide range of influencing factors. Influencing factors of bond can be classified into four main groups:

1. Influences of concrete on bond: concrete compressive strength, concrete tensile strength, concrete cover \(^{39}\), grading curve of aggregate \(^{40}\), consistency of fresh concrete \(^{41}\), fibres in concrete;

2. Influences of reinforcement on bond: relative rib area of bar \(^{41}\), rib pattern of bar \(^{42}\) (including inclination and shape of ribs), diameter of bar, coating (epoxy coating for steel bars or sand coating for non-metallic bars), corrosion of steel bars \(^{43}\), amount of transverse reinforcement \(^{44}\), stress in the reinforcement;

3. Influence of load history on bond: monotonically increasing loading (including influence of loading rate \(^{45}\), long term loading, cyclic loading, reversed cyclic loading \(^{46, 47}\);
4. Influence of position of bar on bond: position of bar during casting, position of bar to the next flexural crack, width of splitting crack along the bar.

Figure 4: Bond development
a) Bond development over an anchorage
b) Bond development overlap a splice
c) Bond-slip relationships as a function of relative rib area

Above parameters influence bond behaviour in different ways. In specific cases careful analysis is required.

Corrosion induced deterioration of reinforced and prestressed concrete structures stimulated research and application of non-metallic reinforcements. Non-metallic reinforcements are made of Fibre Reinforced Polymers (FRP). High strength fibres of FRP can be made of glass, aramid or carbon with a volumetric fibre ratio of 60 to 70%. Matrix resin is usually epoxy resin. Carbon Fibre Reinforced Polymers (CFRP) show excellent fatigue strength, low relaxation and creep behaviour in addition to high tensile strength and corrosion resistance. FRP have tensile strengths of 700 to 3500 N/mm², Young's moduli of 38 000 to 300 000 N/mm², failure strains of 0.8 to 4.0%. FRP show no yielding, behave linearly elastic up to failure with brittle rupture.

Surface treatments (such as spiral fibre winding, indentations, periodic ribs, stranded or braided shape or sand coating) are used to improve bond characteristics of FRP (Figure 5). These treatments can increase bond strength of FRP reinforcements even more than that of steel tendons. During pull-out bond failure of non-metallic reinforcing bars the outer layers of FRP reinforcements (periodic ribs, helical wrapping, indentations, etc.) can be damaged which never occurs in the case of steel tendons or reinforcing bars.
Bond strength and bond behaviour is influenced not only by the concrete properties but also the mechanical as well as surface properties of the non-metallic rebars. These differences may influence structural behaviour.

Deformation capacity of outer layers defines the slip both at bond strength and at failure. Partial damage of ribs can also cause sudden changes of the $\tau_b$-s relationship. Sand coating is resulted in very high adhesion for the reinforcement, however bond strength is reached at limited slip. Residual bond strength is utilised mainly from friction that can be higher or less than in the case of conventional reinforcement depending on the type of fibre, resin and surface configuration. Due to various surface treatments of non-metallic reinforcing bars bond strength can be even higher than that of deformed steel rebars.

New types of concretes (high-strength concrete (HSC), fibre reinforced concrete (FRC) and light weight aggregate concrete (LWAC)) has increasingly become popular within the last few decades.

One of the critical issues is the bond behaviour at high temperatures. During the exposure to high temperatures, concrete undergoes changes in its chemical composition, physical structure and water content. These changes primarily occur in the hardened cement paste. The resulting physical changes and chemical decomposition of major concrete constituents are demonstrated by e. g. cracks, explosive spalling or both $^{51, 52, 53, 54, 55}$.

Investigations on the bond strength between concrete and reinforcing steel at room temperature have been carried out over many years, however, only few experiments are available on the effects of high temperature on the bond characteristics.

Our experimental results on bond between steel and concrete after fire are presented in Ch 2.1.

2 Our experimental studies with bond and anchors at high temperatures

2.1 Bond of anchors after fire

In our study one type of expansion and two types of bonded anchors with adhesives of vinyl ester or vinyl ester with cement were tested in two different concrete grades ($f_{cm}=64.5 \text{ N/mm}^2$, $f_{cm}=43.4 \text{ N/mm}^2$). The effective depth of anchors was 50 mm for a diameter of 8 mm. The maximal heating temperatures were 150 °C and 300 °C, respectively. Reference tests were also carried out on
specimens stored at room temperature. The anchors were installed in concrete blocks of 300x300x100 mm at room temperature, then the specimens were heated from all sides. The temperature was controlled by type K thermo-elements. The specimens were kept for 24 hours on constant maximum temperature to assure uniform temperature distribution in the elements. Pull-out tests were carried out afterwards at room temperature. The pull-out force was measured with dynamometer, the relative displacement was measured by two LVDTs and their signals were averaged.

2.1.1 Torque controlled expansion anchors

In Figure 6 we have illustrated the maximum measured force as a function of the temperature in case of torque controlled expansion anchors (FBN 50+63).

In case of torque controlled expansion anchors we have observed three different failure modes. In the first case we have observed concrete cone failure. In the second case the anchor head lost its ring and we observed pull-out with concrete splitting (small concrete cone). In the third case we observed steel failure at the minimum diameter of the head. This kind of failure did not cause concrete cone failure. The failure mode depended on the concrete strengths and on the temperature. We have observed steel failure of the anchors in case of relatively high strength concrete at 20 °C and also after previous temperature loading of 300°C. In case of lower strengths we have observed steel failure of the anchor only after previous temperature loading of 300 °C.

2.1.2 Bonded anchors

Peak loads of bonded anchors (FIS A 8-175, anchor, FIS V 360 S, vinyl ester mixed with cement, FIS VT 380 C, vinyl ester) as a function of previous temperature loading were demonstrated in Figure 7.

![Figure 6: Peak loads of the torque controlled expanded anchors in function of temperature](image-url)
By comparing the continuous lines in Figures 6 and 7, we can observe similar tendencies of peak load vs. maximal temperature of previous temperature loading up to 300 °C, for torque controlled expansion anchors or bonded anchors using vinyl ester adhesive mixed with cement. However, these bonded anchors provided slightly higher peak loads.

The failure mode depended also on the concrete strengths and on the maximum temperature load. In case of concrete with relatively high strengths we observed concrete cone failure at room temperature after temperature loading to 150 °C and 300 °C. In case of lower strengths concrete we observed shallow concrete cone with bond failure at all test temperatures.

Vinyl ester adhesive is more sensitive to the increase of the temperature. We observed steel failure at 20 °C independent from the bond strength. After heating up to 150 °C we observed different failure modes. In case of higher concrete strength the failure mode was steel failure. In all other cases concrete cone with bond failure was observed. After heating up to 300°C in all cases concrete cone with bond failure and significant decrease of bond strength were observed.
After the pull out tests we analysed the failed bond surface. We did not observe the damage of the adhesive after temperature loading up to 150 °C. After heating up to 300 °C then cooling it down, the adhesive was significantly damaged (Figure 8).

### 2.2 Bond of anchors in fire damaged concrete

In our work we analysed the load bearing capacity of anchors placed in thermally-damaged reinforced concrete. Our primary goal was to assist the reinforcement work of reinforced concrete structural members damaged in fire events. One concrete mixture recipe was used to prepare the specimens. 28-days compressive strength of concrete was measured using 150x150x150 mm cubes. The average compressive strength of the concrete used was \( f_{cm} = 44.79 \text{ N/mm}^2 \). During the experiment, the specimens were exposed to fire load on one side until they reached the desired temperature, then they were allowed to cool down at laboratory temperature (20 °C). The day after the fire load, typically after 24 hours, when the specimen had been cooled down, the fastener was inserted in the thermally damaged specimen. In order to allow the cross-linking of the adhesive, loading of the fasteners took place after a further 24 hours.

During the experiment, anchors have failed in all cases with a concrete cone failure. These failures illustrate that an adhesive bond can be created between the adhesive and the thermally stressed concrete with a strength that caused a concrete cone failure. During the tests, no specimen showed either a clear pull-out failure or the combination of concrete cone failure and pull-out failure. On the surface of the concrete cones, aggregate particles close to the thermally stressed surface had a red-dish discoloration, and the ratio of discoloured particles increased when approaching the embedment depth and with increasing temperature. This discoloration could be explained by the chemical processes that occurred in the quartz gravel. In case of thermally stressed specimens, the crack creating the concrete cone was just running in the cement stone, while aggregate particles remained intact. The aggregate particles could be easily twisted from their positions as a consequence of damage to the adhesion between cement stone and aggregate.

Figure 9 shows the tensile resistance of the anchors, while Figure 10 shows the relative residual resistance values as a function of temperature.

![Figure 9: Relationship between the tensile resistance and the temperature in the embedment depth](image)
We used the tensile resistance of anchors fixed with epoxy resin as a standard; it drops to 49% compared to fasteners that have not been exposed to fire when temperature reaches an average of 200 °C in the embedment depth; it drops to 33% when temperature reaches 300 °C in the embedment depth, and to 32% when temperature reaches an average of 400 °C in the embedment depth.

The force-displacement curves of pull-out tests performed on the standard specimens described a brittle failure (Figure 11). The initial rapid force uptake, after reaching the maximum load, was followed by a rapid failure with small displacements. The force-displacement curves of pull-out tests performed on the thermally stressed specimens showed a gradual decrease in load bearing capacity. It can be observed that the curves are more and more flattened as the temperature increases, which means that failure is accompanied by increasingly greater displacements. It is interesting to note that the maximum recorded force had nearly the same displacement value in all four cases (~1 mm).
2.3 Bond between steel and concrete after fire

Bond behaviour between concrete and reinforcing bars was studied under various levels of elevated temperatures. Five different concrete compositions were used. Hundred five pull-out specimens (Ø120 mm, 100 mm) were prepared. After removing the specimens from the formwork, they were stored in water for seven days then kept at laboratory conditions until testing. The specimens were 28 days old when tested.

We carried out an experimental study to analyse the bond characteristics after being subjected to high temperatures. Test variables were:

- maximal temperature (20 °C, 50 °C, 150 °C, 300 °C, 400 °C, 500 °C, 600 °C, 800 °C)
- type of aggregate (quartz gravel, expanded clay)
- type of fibres (polypropylene fibers, hocked-end steel fibers).

The water cement ratio was constant: w/c=0.43. The amount of cement, water, aggregate, fibres and plasticizer are given in Table 1. The consistency of concrete was measured by flow table tests and resulted 450 to 500 mm.

| Table 1: Experimental concrete mixes (* polypropylene fibers, **hocked-end steel fibers) |
|---------------------------------|-----|-----|-----|-----|-----|-----|
| concrete (kg/m³) | Mix 0 | Mix 1 | Mix 2 | Mix 3 | Mix 4 | Mix 5 |
| cement (kg/m³) | 350 | 350 | 350 | 386 | 386 | 350 |
| water (kg/m³) | 151 | 151 | 151 | 181 | 181 | 151 |
| aggregate (kg/m³) 0-4 mm | quartz sand | 912 | quartz sand | 912 | quartz sand | 912 |
| aggregate (kg/m³) 4-8 mm | quartz gravel | 485 | quartz gravel | 485 | expanded clay | 302 |
| aggregate (kg/m³) 8-16 mm | quartz gravel | 544 | expanded clay | 544 | quartz gravel | 485 |
| plasticizer kg/m³ | 1.4 | 1.4 | 1.4 | 5 | 5 | 1.4 |
| fibres (kg/m³) | - | 1* | 1* | - | - | 35** |

The pull-out specimens had a diameter of 120 mm and height of 100 mm. Slip was measured with two LVDTs at the unloaded side.

Figure 12 indicates the measured relative residual bond strength values of concrete as a function of maximal temperatures up to 800 °C. Pull-out test were carried out at room temperature after heating and cooling the specimens. The following conclusions can be drawn:

1. The relative bond strength reduction was higher than the relative compressive strength reduction in all cases.
2. Most considerable reduction of bond strength took place between 400 °C and 500 °C. This reduction can be explained by the decomposition of portlandite at 450 °C.
3. The relative bond strength of lightweight concrete with expanded clay (Mix 2 and Mix 3) was higher up to 400 °C but lower above 500 °C compared to concrete with quartz gravel aggregate.

4. The relative bond strength of fibre reinforced concrete (Mix 1 and Mix 2) was lower up to 400 °C but higher to 500 °C and higher temperatures as in the case of concrete with quartz gravel aggregate.

Figure 12: Test result on bond strength measured on ribbed reinforcement (each point gives the average of 3 measurements)

Bond strength–slip diagrams for Mix 0 as a function of temperature are represented in Figure 13.

The following conclusions can be drawn:

1. With increase of temperature the bond stress decreases and the slip values increase.

2. After 20 °C, 50 °C and 150 °C temperature loading, the bond strength-slip diagrams show the same tendencies. The strength reduction is less than 20 %.

3. After 600 °C and 800 °C temperature loading, the tendencies of bond strength-slip diagram change. This could be explained by the missing of chemical bond (decomposition of portlandite). The strength reduction is 80 % after 600 °C temperature loading, and 93 % after 800 °C temperature loading.
Conclusion

Interaction between structural elements or structural materials can be provided by fastening elements or directly by bond, respectively. Fastening elements and bond have to fulfil multiple requirements on force transfer capacity, deformation capacity, ductility, durability, fire resistance in addition to easy application.

Present paper intended to review some of the new potentials in anchoring solutions as well as for bond. New types of concretes as well as new types of reinforcements provide new technical solutions as well as characteristics that have to be controlled.

Specific parts of this paper give new test results on bond of anchors as well as bond between steel reinforcement and concrete after fire:

**Bond of anchors after fire:** The anchors were installed in concrete blocks which were previous heated up to 150 °C or 300 °C. Reference tests were also carried out on specimens stored continuously at room temperature (20 °C). The failure mode depends in all cases on concrete strengths and the maximal previous temperature. Torque controlled expansion anchors and bonded anchors using vinyl ester adhesive mixed with cement have similar tendencies of peak loads vs. maximal temperature of previous temperature loading up to 300 °C. Vinyl ester adhesive is more sensitive to the increase of the temperature. The peak loads after the previous temperature loading up to 300°C were significantly reduced by bonded anchors using vinyl ester adhesive.

**Bond of anchors in fire damaged concrete:** In our work we analysed the load bearing capacity of anchors placed in thermally-damaged reinforced concrete. Our primary goal was to assist the reinforcement work of reinforced concrete structural members damaged in fire events. The load capacity of anchors created with epoxy adhesive decreased with increasing temperature during thermal loading. When plotting the tensile resistance in function of the temperature, it can be said that:

- if temperature reaches 200 °C in the embedment depth, then tensile resistance drops to 49%,
- if temperature reaches 300 °C in the embedment depth, then tensile resistance drops to 33 %,
- if temperature reaches 400 °C in the embedment depth, then tensile resistance drops to 32 %,
During the investigation we found no delamination (spalling) of the concrete in any of the specimens, so the results of the test can be used only in cases where spalling does not occur in the reinforced concrete structure during fire.

**Bond between steel and concrete after fire:** The following conclusions can be drawn from our experimental study on the influence of high temperatures to the residual bond characteristic. Pull-out specimens tested at cold state after heated up to (20 °C, 150 °C, 300 °C, 400 °C, 500 °C, 600 °C and 800 °C). The types of concrete were: C, SFRC, PPRC, LWAC1, LWAC2. Type of steel reinforcement was deformed rebar. Most considerable reduction of bond strength took place between 400 °C and 500 °C in all cases. This reduction can be explained by the decomposition of portlandite by 450 °C. This was valid for all tested concrete types (C, SFRC, PPRC, LWAC1, LWAC2) with all tested reinforcements (deformed rebar). Reduction of bond strength both below 400 °C and above 500 °C are close to linear on different levels.

4 **Acknowledgement**

The second author acknowledges the support by the János Bolyai Research Scholarship of the Hungarian Academy of Sciences.

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RECENT DEVELOPMENTS IN COMPOSITE STRUCTURES

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ABSTRACT

Composite steel and concrete structures represent an advantageous construction method since it combines steel and concrete using the capacity of the materials in an optimum way. In recent years, the use of composite structures has undergone a wide spread. Its application in office buildings, parking grounds or bridges is becoming more common. In the last years, several research projects within the area of composite structures have been completed successfully. New design concepts were developed, which in most of the cases are going to be implemented for standardization.

This paper is a review of the latest developments in steel concrete composite structures. Special progress has been achieved in view of composite connections between steel and concrete where new systems such as concrete dowels have been developed5,6. Innovative findings for composite beams with trapezoidal steel sheetings25 and horizontally lying studs44 were discovered. Extensive research in the field of slim-floor beams has led to design rules with economical solutions for composite flooring systems with small slab thicknesses21, 22, 33. Within a holistic conceptual view, design models for the behavior of composite joints have been developed based on the component method in order to capture the overall behavior of the composite structures27, 29, 30, 32. Composite bridges profit from recent approaches in view of lifecycle behavior and sustainability26. Fire behavior has become a topic especially for the design of composite columns1, 41.

Besides the elaboration of design rules an overview about the normative development within the 2nd generation of Eurocodes is given. In the frame of a systematic review of EC 4 improvements towards the simplification of design rules, harmonization with other standards such as EC 2 and EC 3 and the introduction of new construction methods are described.

1 Introduction

Different aspects of composite structures are outlined in the present paper. This includes the description of current normative developments for the 2nd generation of Eurocodes. More specific tasks are described which will be realized by Project Teams in the frame of the so-called Mandate 5159 to further develop Eurocode 4 (EC4) Design of composite steel and concrete structures. In a second part, an overview is given on a number of research issues, which support the code revision and reflect well the recent tendencies in steel concrete composite structures without claiming completeness.
2 Normative developments and implementation of recent research results

2.1 General

The Eurocodes were developed to enable the design of structural construction works, buildings and civil engineering works, on a harmonized European level. All 10 of the existing Structural Eurocodes from Actions on structures (EC1) to Design of Concrete (EC2), Steel (EC3) and Composite steel and concrete structures (EC4, 16, 17, 18) up to Design of structures for earthquake (EC8), in altogether 58 parts, were published prior to June 2007. Their development was a great achievement and represented the culmination of over 30 years’ collaborative effort. Their impact has been considerable, affecting the day-to-day work of around 500 000 professional engineers across Europe. In the Eurocodes, in order to allow countries to decide on own safety levels and to give national geographic and climatic data so-called Nationally Determined Parameters (NDPs) are open for choice in the frame of National Annexes. As a consequence, the full implementation of the Eurocodes in the European countries needed until 2010 when all national codes had been withdrawn and replaced by the Eurocodes and belonging National Annexes.

It is widely recognized that long-term confidence in the codes requires the Eurocodes to evolve in an appropriate manner. The meanwhile accepted work program for the 2nd generation of Eurocodes focuses on ensuring the standards remain fully up to date through embracing new methods, new materials, and new regulatory and market requirements. Furthermore, it focuses on further harmonization and a major effort to improve the ease of use of the suite of standards for practical users. In order to show opportunities for participation in the development of these new design rules, the normative process is explained in detail in the following.

2.2 Mandate 5159 – Development of 2nd generation of Eurocodes

2.2.1 Overview

The further evolution of the Eurocodes is planned in the frame of the mandate M/5159, which was agreed in December 2012 between the European Commission and CEN. Among others, aims of the mandate are the extension of the Eurocode rules in terms of new materials, products and construction methods, improving the practical use for day-to-day calculations and achieving a better harmonization by reducing the number of NDPs.

The first revised version of the Eurocodes was originally planned to be published in 2020. However, some organizational matters in the negotiation process between the European Commission and CEN have meanwhile caused a delay of about 1 year. Figure 1 shows the time schedule for the revision and the further evolution of the Eurocodes.
In general, the revision can be subdivided in the following two activities:

**General revisions and maintenance of the Eurocodes:** This is the usual procedure for a code revision according to CEN, which is launched in form of a call for ‘systematic reviews’ to the national standardizing bodies. The evaluation and implementation of the suggestions and comments is then carried out by the CEN Subcommittees and Working Groups.

**Technical enhancements of the Eurocodes in the frame of the Mandate M/515:** The further development takes place simultaneously to the general revision within Mandate M/515. Similar to the transfer of the ENV-versions into the EN-version, the realization is conducted by Project Teams (PT) that consist of a maximum of six members.

Figure 1: Planned time-table for the revision of the Eurocodes

The CEN/TC 250 work program has been split into four overlapping phases, as illustrated in Figure 2. This has been done to enable the interdependencies between activities to be effectively managed, and ensure that the work is undertaken as efficiently as possible. Phase 1 includes those parts of the work program upon which other activities are primarily dependent for reasons of overall coordination, technical scope or because they are essential for achieving the target dates for delivery of the next generation of Eurocodes. Phase 1 of the mandate started in 2015 and will end beginning of 2018 after a 3-years term. Phase 1 consists of a volume of about 4.5 million Euros. Phase 2 will start in this year summer also for a term

Figure 2: Indicative phasing of the work
of 3 years. For Phase 3 and Phase 4 negotiations have started between the European Commission and CEN. First experiences with the project teams’ work and the evaluation of the systematic reviews show an enormous need of further development and harmonization.

2.2.2 Main aims
An important task of the program is to reduce the number of Nationally Determined Parameters (NDPs). Table 1 gives a summary of the number of NDPs in the current Eurocodes, relative to the number of parts in each Eurocode and its number. The very uneven distribution also show that for some Eurocodes NDPs form a means to overcome different views on technical items. In these cases, the document N1250\textsuperscript{10} recommends a procedure to overcome these differences in order to reach a better harmonization.

Table 1: Analysis of NDPs in the current Eurocodes\textsuperscript{10}

<table>
<thead>
<tr>
<th>Eurocodes</th>
<th>No. of parts</th>
<th>No of pages</th>
<th>No of NDP’s</th>
</tr>
</thead>
<tbody>
<tr>
<td>EN 1990</td>
<td>1+Annex A2</td>
<td>90+30</td>
<td>54</td>
</tr>
<tr>
<td>EN 1991</td>
<td>10</td>
<td>770</td>
<td>292</td>
</tr>
<tr>
<td>EN 1992</td>
<td>4</td>
<td>450</td>
<td>176</td>
</tr>
<tr>
<td>EN 1993</td>
<td>20</td>
<td>1250</td>
<td>236</td>
</tr>
<tr>
<td>EN 1994</td>
<td>3</td>
<td>330</td>
<td>42</td>
</tr>
<tr>
<td>EN 1995</td>
<td>3</td>
<td>225</td>
<td>21</td>
</tr>
<tr>
<td>EN 1996</td>
<td>4</td>
<td>300</td>
<td>31</td>
</tr>
<tr>
<td>EN 1997</td>
<td>2</td>
<td>340</td>
<td>42</td>
</tr>
<tr>
<td>EN 1998</td>
<td>6</td>
<td>600</td>
<td>103</td>
</tr>
<tr>
<td>EN 1999</td>
<td>5</td>
<td>500</td>
<td>58</td>
</tr>
</tbody>
</table>

As a second point “Enhancing ease of use” has been defined as a major aim of the development of the 2\textsuperscript{nd} generation of Eurocodes. A number of principles and related priorities have been defined after long discussions in TC250, the responsible committee for Structural Eurocodes in CEN, see Table 2.

Table 2: Principles and related priorities\textsuperscript{10}

<table>
<thead>
<tr>
<th>General principles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Improving clarity and understandability of technical provisions of the Eurocodes</td>
</tr>
<tr>
<td>2 Improving accessibility to technical provisions and ease of navigation between them</td>
</tr>
<tr>
<td>3 Improving consistency within and between the Eurocodes</td>
</tr>
<tr>
<td>4 Including state-of-the-art material the use of which is based on commonly accepted results of research and has been validated through sufficient practical experience</td>
</tr>
<tr>
<td>5 Considering the second generation of the Eurocodes as an “evolution” avoiding fundamental changes to the approach to design and to the structure of the Eurocodes unless adequately justified</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Specific principles (secondary)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 Providing clear guidance for all common design cases encountered by typical competent practitioners in the relevant field</td>
</tr>
<tr>
<td>7 Omitting or providing only general and basic technical provisions for special cases that will be very rarely encountered by typical competent practitioners in the relevant field</td>
</tr>
<tr>
<td>8 Not inhibiting the freedom of experts to work from first principles and providing adequate freedom for innovation</td>
</tr>
<tr>
<td>9 Limiting the inclusion of alternative application rules</td>
</tr>
<tr>
<td>10 Including simplified methods only where they are of general application, address commonly encountered situations, are technically justified and give more conservative results than the rigorous methods they are intended to simplify</td>
</tr>
<tr>
<td>11 Improving consistency with product standards and standards for execution</td>
</tr>
<tr>
<td>12 Providing technical provisions that are not excessive sensitive to execution tolerances beyond what can be practically achieved on site</td>
</tr>
</tbody>
</table>

2.2.3 Eurocode 4
The work within the mandate of the 3 parts of EC 4 has been subdivided into 8 tasks given in Table 3 by the SC4, the responsible subcommittee for EC4, also the phase of realization is indicated. Each of the tasks is realized by one Project Team.
Table 3: Tasks of Mandate M/515 for EC4

<table>
<thead>
<tr>
<th>Task Ref.</th>
<th>Task Phase</th>
<th>Corresponding Part of EN 1994</th>
<th>Task name</th>
</tr>
</thead>
<tbody>
<tr>
<td>SC4.T1</td>
<td>Phase 1</td>
<td>EN 1994-1-1$^{16}$, EN 1994-1-2$^{17}$, EN 1994-2$^{18}$</td>
<td>Respond to demands from industry, including needs for harmonization with EC 2 and EC 3</td>
</tr>
<tr>
<td>SC4.T2</td>
<td>Phase 1</td>
<td>EN 1994-1-1$^{16}$, EN 1994-1-2$^{17}$</td>
<td>Composite beams with large web openings</td>
</tr>
<tr>
<td>SC4.T3</td>
<td>Phase 1</td>
<td>EN 1994-1-1$^{16}$</td>
<td>Revised rules for shear connection in the presence of modern forms of profiled sheeting</td>
</tr>
<tr>
<td>SC4.T4</td>
<td>Phase 1</td>
<td>EN 1994-1-2$^{17}$</td>
<td>Development of new rules for composite columns in fire</td>
</tr>
<tr>
<td>SC4.T5</td>
<td>Phase 2</td>
<td>EN 1994-1-1$^{16}$, EN 1994-1-2$^{17}$</td>
<td>Development of rules covering shallow floor construction and other flooring types using precast concrete elements</td>
</tr>
<tr>
<td>SC4.T6</td>
<td>Phase 4</td>
<td>EN 1994-1-1$^{16}$</td>
<td>Finalization of the evolved EN 1994-1-1$^{16}$</td>
</tr>
<tr>
<td>SC4.T7</td>
<td>Phase 4</td>
<td>EN 1994-1-2$^{17}$</td>
<td>Finalization of the evolved EN 1994-1-2$^{17}$</td>
</tr>
<tr>
<td>SC4.T8</td>
<td>Phase 4</td>
<td>EN 1994-2$^{18}$</td>
<td>Finalization of the evolved EN 1994-2$^{18}$</td>
</tr>
</tbody>
</table>

Task 1 – Phase 1

Task 1 of SC4 includes a general revision of all three parts of EC 4 and focuses mainly on the reduction in number of National Choices (NDPs), the enhanced ease of use by identification and analysis of paragraphs, clauses and formulae that require simplification, clarification, correction or harmonization. Therefore, approx. 250 comments from national bodies collected by the systematic review are mainly considered and now compiled into modified rules.

Task 2 – Phase 1

Within this task, a new Annex or part of EN 1994 for the design of composite beams with large web openings that are widely used across Europe is generated. At the same time a new Part 1-13 for Eurocode 3 on steel girders with large web openings is developed. New generic design procedures are needed to ensure that all these solutions satisfy certain general principles, and to give designers of such constructions simple and clear procedures to follow.

Task 3– Phase 1

Revised rules for shear connections with modern forms of profiled sheeting are highly demanded in practice. Therefore, revised push-out tests, new design procedures for shear connectors with modern forms of profiled sheetings are developed in order to be implemented in the new version of EN 1994-1-1$^{16}$. Current research results on this topic are given in Chapter 4.2.

Task 4– Phase 1

Design procedures to better predict the load bearing resistance of concrete filled tubes in fire are developed for replacing existing EN 1994-1-2 Annex H$^{17}$. Current research results on this topic are given in Chapter 4.4.

Task 5– Phase 2

Within Task 5 new rules covering shallow floor construction and other flooring types using precast concrete elements are developed. Current research results on this topic are given in Chapter 4.3.

Task 6 to 8– Phase 3 or 4

In these tasks, the three different parts of EC 4 should be finalized. Relevant output from all previous tasks should be incorporated and harmonized with new rules in the second generation of EC3 for steel and EC2 for concrete structures.
3 Current research on composite structures

3.1 General
In the following chapter an overview on current research on composite structures on a European level is given. Due to the limited space, this contribution makes no claim to completeness. However, some tendencies of recent developments within steel-concrete composite structures are shown, in most cases with relation to the ongoing normative developments.

3.2 Composite connections

3.2.1 Concrete dowels
Concrete dowels presenting an alternative to the standardized headed studs are shear connectors consisting of recesses in steel girders introduced into the concrete slab. Through recesses that can be of different geometrical shape reinforcement bars are led as transverse reinforcement. The development that started during the 90s of the former century by the ‘Perfobond-Leiste’ by Leonhard and also had been applicable by an approval nowadays creates new optimized shapes that also initiate new forms of beam structures. Recently, German national approvals were admitted for the concrete dowels that are puzzle or clothoid shaped as indicated in Figure 3. Furthermore, the approved Composite Slim-Floor Beam-System (CoSFB) consists of a commercially available I- or H-profile with introduced holes in the web working as concrete dowel. The introduction of the shear connectors in the web of the steel profile enables to reduce the constructional height of the construction to a minimum, what makes it very attractive especially for the use in Slim-Floor Constructions.

Figure 3: a) Puzzle and clothoid shaped concrete dowel, b) + c) Examples of applications for puzzle and clothoid shaped concrete dowels²

The shear connection behavior of concrete dowels is defined by a three-axial pressure condition. To prevent the puzzle or clothoid shaped shear connections from failing by longitudinal shearing the following three failure modes have to be considered²⁰:

Concrete shearing: The shear area and the shear strength of the concrete dowel are decisive.
Concrete pry-out: Breakaway of the concrete between the shear connector and the concrete surface.
Steel failure: Failure may occur due to a combination of small plate thickness, low steel strength and high concrete strength.
Comparison with headed studs shows the competitiveness of concrete dowels in terms of shear capacity (see Figure 4)\textsuperscript{20}.

![Figure 4: Comparison of shear resistance: headed studs - concrete dowel\textsuperscript{20}](image)

For the Composite Slim-Floor System (CoSFB) failure due to concrete pry-out is prevented by the top flange of the steel profile. Main parameters, which influence the strength, are concrete strength, the diameter of the reinforcement bars and the diameter of the holes in the web. The shear load capacity is achieved by a combination of three-axial concrete pressure in the holes of the profile and a bending or shearing of the reinforcement bars\textsuperscript{8,32}.

![Figure 5: Composite Slim-Floor System (CoSFB)\textsuperscript{6,8,32}](image)

### 3.2.2 Headed studs arranged close to the concrete edge

Headed studs arranged close to the concrete edge present a limited carrying capacity because of the splitting forces induced in the concrete, which lead to an earlier failure as for headed studs positioned without any edge influence\textsuperscript{44}, see Figure 6. The ultimate resistance and the fatigue strength of horizontally lying studs or, better, headed studs close to the concrete edge are strongly influenced by the effective edge distance $a_{e'}$ (see Figure 7). Therefore, not the horizontal or vertical arrangement of the shear studs is of importance but the distance to the concrete edges. Therefore, the title of the Annex in EC4-2, which is at the moment named “Headed studs that cause splitting forces in the direction of the slab thickness” should be changed accordingly when taken over in a new version of Eurocode 4. It is also planned to transfer this content of the existing Annex C of EN 1994-2\textsuperscript{18} to the future EN 1994-1-1\textsuperscript{16}, since the rules are valid for buildings as well.
Some improvements have been suggested for the conversion of the existing Annex C of EC 4-2 for bridges into the new version of EC 4-1-1 that is under development. Part of the enhancement consists of simple clarifications, part in technical changes due to research, which has been realized in the meantime. That is the case of the fatigue behavior under transverse shear loading for which design rules are introduced, similar to what has been included for longitudinal shear. Some open questions are still left, for instance, the interaction of longitudinal and vertical shear or also the geometric restrictions to prevent pull-out failure for horizontally lying studs in edge position. For the latter the experience from fastening technique, EN 1992 has started to be considered. These new developments now allow a design for both static and fatigue loading in longitudinal and vertical direction and the application of interesting new composite sections, see Figure 8.

3.2.3 Shear connections in composite girders with corrugated web

One of these innovative composite bridge girders with horizontally lying headed studs has been researched at the University of Stuttgart. The design of the section consists of a corrugated steel web where the upper flange has been replaced by the concrete slab connected by headed studs welded to the web (see Figure 8). The longitudinal and vertical shear strength of the connection provided by the corrugation together with the studs has been focused in the studies. For the case of one layer of studs, experimental and numerical investigations have given as a result the formulation of an equation, where the concrete resistance, the length of the steel embedment and the friction coefficient are the most important parameters. A further evolution of the section is introduced by using double layers of studs and a prefabricated concrete slab, the shuttering is no longer necessary what means a reduction in times of construction and a better final appearance (see Figure 9).
3.2.4 Demountable shear connectors

In consideration of the full lifecycle of products, recently also buildings come to the fore, where demountable shear connectors form a good basis. As the minimizing of costs and materials during the construction phase is part of research since the beginning, also demolition and recycling are now part of research. To separate the steel beam and the concrete slab easily, demountable shear connectors were developed that can be attached on a steel girder like a screw with a nut (see Figure 10). When the building is going to be demolished, the nut will be released, what enables to remove the concrete decking easily as indicated in Figure 11. Push-out tests showed an equivalent ductility with a lower initial stiffness compared with welded headed studs\textsuperscript{34,45}.

![Figure 10: Demountable shear connector\textsuperscript{34,45}](image)

Figure 10: Demountable shear connector\textsuperscript{34,45}  

![Figure 11: Disassembly of composite beam with demountable shear connector\textsuperscript{34}](image)

Figure 11: Disassembly of composite beam with demountable shear connector\textsuperscript{34}

3.3 Composite slabs and beams

3.3.1 Composite beams with profiled steel sheeting

Composite beams with profiled steel sheeting have to satisfy the ductility criteria of the connection to allow for the calculation of the full plastic bending resistance according to EC 4. New developments of profiled steel sheeting have deep decks and narrow ribs with wide distances between each other. By this geometry, it is difficult to achieve the required degree of shear connection, even by placing shear connectors in every rib. Research on composite beams with deep deck profiled steel sheeting and accordingly a degree of shear connection smaller 40 \%\textsuperscript{16} was conducted\textsuperscript{7} (see also Figure 12). Experimental and numerical investigations on composite girders with deep decking showed that a determination of the plastic moment according to EC 4 is still possible for smaller degree of shear connection due to the extremely ductile behavior. New methods were developed considering the behavior of headed studs in the ribs even for this new generation of profiled steel sheeting\textsuperscript{38,25,33}. A good estimate of the connection strength is the prerequisite of a good prediction of the beam behavior. Furthermore, the reduced degree of shear connection leads to higher midspan deflection of the beams. Instead of ULS (Ultimate Limit state) the design of the beam may be controlled by SLS\textsuperscript{33} (Serviceability Limit state) criteria.
3.3.2 Slim-floor constructions

Slim-floor constructions are composite beams with integrated steel profiles in the concrete slab and an overall construction height as high as the profile or slightly higher. The resulting good fire resistance, control of human induced vibrations, acoustic performance, thermal capacity and the stiffness make these kind of constructions so attractive and competitive to conventional concrete solutions. The shallow geometry subjects slim-floors to nonlinear effects of cracking. While the design procedures in ULS are in accordance with EC 4, the bending capacity of the concrete slab needs to be considered particularly for calculation of realistic deformations (SLS). Therefore, calculation methods were developed taking these nonlinear effects into account. For an implementation in the next version of EC 4 design and construction rules and lifecycle assessments are developed in the frame of a recent research project supported by intensive testing, see Figure 13.

Figure 13: Experimental investigations on single span slim-floor construction (left), part of cross-section (right)

3.4 Composite columns

3.4.1 Concrete Filled Hollow sections exposed to fire

The advantages of concrete filled tubular columns in terms of capacity, ductility and functionality positioned them as an attractive structural solution and has contributed to their application spread. Another important aspect that has driven its use forward is the good fire performance. Nonetheless, the simplified method of Annex H in EN 1994-1-2 to calculate the fire resistance in concrete filled hollow sections had to be limited to relative slenderness under 0.5 due to safety relevant discrepancy.
cies to FE results. The research has focused on the extension of the application range of the method in Annex H. An important drawback for the users of the method is the temperature calculation. The European project FRISCC “Fire Resistance of Innovative and Slender Concrete Filled Tubular Composite Column” financed by RFCS funds has been devoted to overcome these shortcomings, so that the method can extend to higher slenderness and new column shapes. There are two main lines of work, one to characterize the temperature of the columns by means of a simplified cross-sectional temperature field. The second part is the determination of the ultimate buckling load in fire situation, but extending the range of application to most of the situations presented in the reality.

3.4.2 Fire behavior of blind-bolted connections to concrete filled tubular column

Although there are aspects related to the concrete filled tube (CFT) column behavior that still have to be covered and implemented in the Eurocodes, some knowledge is more advanced than in the case of typical beam-to-column joints of I-sections. The component method in EN 1993-1-8 does not give any rules to consider these columns, since the behavior is not characterized for all the components. In that direction, there are different projects dedicated to give the guidelines to design CFT column joints under fire taking into account resistance, stiffness and ductility. A characteristic of the connections to hollow columns is that in the case of bolted joints they require blind bolt systems able to be bolted from one side of the column. The fire performance of blind-bolted connections to CFT was studied numerically in the tension zone of moment-resisting joints. The objective was to gain insight into tensile loaded blind-bolted connections in fire and to assess the effect of concrete infill and the anchorage of the blind-bolt. Two types of connections were analyzed, a single blind-bolt connecting a plate to a rectangular hollow steel section (RHS) column and a double T-stub connection to a RHS column (see Figure 14). The type of column (Hollow steel section (HSS) and CFT) and the type of fastener system (Hollo-bolt and extended Hollo-bolt) were the variables considered for each connection. For a relative thick plate and RHS column, the fastener system (sleeve or the bolt shank) governs the failure; nonetheless, a larger parametrical study should be accomplished.

Figure 14: T-stub blind-bolted connection at the tension zone of the endplate connection
3.5 Composite joints

Due to a gap between the design of fastenings in concrete and steel design and missing standardized joint solutions, for the designers it is often the easiest solution to realize the whole structure in concrete, although according to the performance of the materials an innovative steel-to-concrete solution would be the better choice. Within several research projects, mechanical models for these joints (see Figure 15) were developed based on the component method in order to simplify the design rules.

The original concept for anchor plates based on the component method in existing EC3 Part 1-8 distinguishes between different failure modes based on a characterized structural behavior of the single components, mainly on the steel side. Meanwhile, in a new overall design concept steel components according to EN 1993-1-8 and concrete components according to EN 1992-4 are considered simultaneously. In addition to the load-carrying capacity of the joint, the load-deformation behavior can be determined for the whole joint also for the concrete components.

Thereby, in an optimized design of the joint, it is possible to strengthen certain, more flexible components, thus achieving ductile behavior of the whole joint. For example, if the thickness of the anchor plate is varied, the flexible behavior of the anchor plate, which is considered within the T-stub under tension and compression according to EN 1993-1-8, can come into play. On the other hand, higher resistances for some concrete failure modes can be reached, if, for example, supplementary reinforcement is placed next to the headed studs in tension and the interaction between concrete and reinforcement is considered according to recent approaches.

Besides the development of new design rules for these composite joints, the structural behavior of these joints is a key aspect, if the robustness of composite frame structures is under consideration. Nowadays the demand for a robust design of structures is increasingly important. In general, the probability of an accidental event is small, but the consequences are often fatal. Robustness does not necessarily mean to mitigate a local damage of the structure, but to mitigate a progressive collapse of the entire structure due to this local damage. One possibility to fulfil the robustness requirements is to accept the local damage, as for example a loss of a column, and to activate alternate load paths in order to redistribute the available and additional loadings into the intact part of the structure. When applying the alternate load path method for redistributing the loads resulting from a column loss especially the detailed consideration of the behavior of beam-to-column joints is important (see Figure 16). Among others, it has to be taken into account that the joints are not only exposed to a hogging...
bending moment (joint adjacent to column loss), but due to the column loss also to sagging bending moment at the joint (joint directly above column loss). Moreover, due to the load redistribution catenary action may develop, if the joints are able to carry membrane forces. A crucial aspect is the investigation of the available rotation capacity of the joints which can be described as well by the component method according to EN 1993-1-8[15]. In contrast, the required rotation capacity of the joints is determined by the consideration of the system. A design strategy as well as a detailed definition of the joints that can be used in order to verify the rotation capacity for accidental loading situations has been developed on a European level[12, 30, 30, 46].

3.6 Composite bridges

3.6.1 Integral bridges

The concept of integral bridges refers to bridges where superstructure and substructure (piles and abutments) are monolithically connected. The advantages of this type of bridges comes from the distribution of the bending moments that are reduced in the midspan and allows slender cross section with the subsequent savings of materials, production, transport and construction. The other positive aspect is that bearings and expansion joints are avoided, instead, there is a continuous transition from the solid ramp to the superstructure. Therefore, the problems of inspection and maintenance of the bearings are suppressed.

These integral abutments are mainly used in composite bridges of one span, they are founded on piles or footings. The abutments have to be designed to bear not only the vertical shear forces but also the horizontal forces and the bending moments. The design of the interaction soil abutment to define the rigidity is crucial. The insight in the behavior of the connection is objective of current researches which has defined guidelines[2]. The use of these types of bridges is also associated with reduced times of construction and therefore is linked to the choice of prefabricated composite girders sections that contributes to a fast erection of the bridge. An advantageous scheme of construction has been applied for a number of these bridges: having prefabricated composite girders mounted during a short interruption of the traffic and placed on simple provisional bearings, the encasement of the girders is cast at the same time as the abutment is finalized. Consequently, the cast in situ of the final layer of the concrete slab of the bridge is already realized in an optimal way at an encased system.
3.6.2 Holistic consideration of the sustainability on steel-concrete composite road and railway bridges

In the recent years, an important aspect of research in composite bridges is sustainability, for being a key issue for the design of constructions. Sustainability is especially relevant for bridges because of its service life of 100 years and the fact that all the life-cycle stages need to be considered. That means at the tender stage decision making should no longer be only based on construction costs but on a holistic assessment of sustainability aspects along the whole lifespan of the bridge.

In the European research project "SBRI" and the German research project "NaBrüEIS" a holistic approach of the sustainability on bridges has been developed, where environmental quality (LCA Life-Cycle Assessment), economical costs (LCC Life-Cycle Costing) together with external effects were considered. In the SBRI project\(^2\), different type of road bridges and variants of them were considered. Valuable knowledge has been gathered in an extensive databank considering especially realistic data for inspection and maintenance. A useful software tool has been developed, integrating the databank and a methodology of enrichment evaluation by weighting the criteria. This knowledge is currently in process of dissemination, since until now was only accessible to interested experts, by means of the dissemination project SBRI+\(^4\).

Meanwhile, NaBrüEIS\(^2\) deals with railway bridges, which are dominated by their embedment into a railway network with interdependencies. A difference between the road bridges and the railway bridges are the external effects due to the traffic interruption that play a crucial role on the holistic approach of the bridge. In road bridges, the external effects relate to the impact on the traffic flow in terms of user costs whereas in railway bridges, the external effects refer to the operation encumbrance costs linked to the operation of the railway net. Additionally, railway bridges have different loadings, which have a stronger impact in view of fatigue, and that also explains the valuable knowledge gained for the fatigue design\(^2\).

It might be emphasized that a lifecycle analysis is not a decision making approach in itself; but it provides valuable information for decision makers in the process of development and selection. Therefore, the easy access to these methods is key for a sustainable bridge design for designers and planners as well as bridge authorities.
4 Summary and Outlook

Within this paper, the normative evolution for composite structures in the frame of the development of a second generation of Eurocodes is shown. Thereby, the revision process of all three parts of EN 1994 in the frame of Mandate M/515\(^9\) is described. Besides the general revision and maintenance of the Eurocodes the technical review of some selected outcomes of former and current research projects on composite structures is given. New tendencies of performance requirements also on composite structures are explained. This includes in addition to improved economical and functional aspects new approaches with regard to robustness, durability and sustainability. Composite structures form competitive structural solutions that are well equipped for the future also due to diverse international research activities and common harmonized efforts for the implementation in the future codes.

References:


CODES AND REGULATIONS
DESIGN OF FASTENINGS FOR USE IN CONCRETE
CONSTRUCTION: NEW EN 1992-4 — CURRENT STATUS,
COMMENTARY AND BACKGROUND

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ABSTRACT

EN 1992-4 has been derived from the revision of CEN/TS 1992-4 which comprises a general first part plus four additional, product specific parts for headed fasteners, anchor channels, mechanical and chemical post-installed fasteners. The revision aimed at shortening the text of CEN/TS 1992-4 in line with standard specifications, thus enabling the regulations for designing the anchorage of fastenings in concrete to be incorporated in part 4 of Eurocode 2 (EN 1992). Furthermore, all types of fastenings are contained in a single document.

To represent the generally recognized state of the art, EN 1992-4 has been expanded to include important special subjects. These subject areas have either been included or summarized in so-called Technical Reports (TR). EN 1992-4 is currently undergoing approval and publication process.

The large product variety and continuously increasing range of applications require the designers and users to acquire an ever-increasing amount of specialist knowledge in order to be able to make the best use of such anchorages. It is in particular the design process that has become significantly more complex in recent years.

For these reasons, the German Committee for Structural Concrete (Deutscher Ausschuss für Stahlbeton = DAfStb) decided as early as in the editing stage of EN 1992-4 to develop a commentary on EN 1992-4. The explanations given in DAfStb Heft 615 are intended to help designers to understand and use EN 1992-4 by providing comments and presenting scientific foundations. The commentary additionally published in English to promote its acceptance by a large expert community in Europe.

In this paper, the main developments within EN 1992-4 will be explained. Furthermore, the DAfStb-commentary book will be introduced and discussed in detail.
Development of the design guidelines

In 1997 the Guideline for European Technical Approval of Metal Anchors for use in Concrete (ETAG 001) was published. It is the basis for granting European Technical Approvals (ETAs, ETZs). After several revisions, the guideline (June 2013) consists of six parts and four annexes.

A generally valid design method has been developed for metal dowels, which is published in Annex C of ETAG 001. This method is based on the standard design and safety concept based on partial safety factors in the ultimate and serviceability limit states. Annex C is supplemented by various Technical Reports (TR), which include a further developed design method for bonded anchors (TR 029), for metal anchors under seismic actions (TR 045) and for anchor channels under static loading (TR 047) and under fatigue loading (TR 050).

In 2009, the Technical Specification CEN / TS 1992-4 "Design of fastenings for use in concrete", German version DIN SPEC 1021-4, was published as a pre-standard. It consists of a general first part and further four product-specific parts for headed studs, anchor channels, mechanical and bonded anchors. In contrast to ETAG 001, Annex C, the pre-standard is also valid for cast-in systems such as headed studs and bolts and anchor channels.

EN 1992-4 was derived from the revision of CEN / TS 1992-4. The aim of the revision was to reduce the content of the CEN / TS 1992-4 so that the rules for design of fastenings in concrete could be transferred to Part 4 of Eurocode 2 (EN 1992).

In EN 1992-4 there are some opening clauses with the possibility of national regulations. In the German National Annex to DIN EN 1992-4, most of the recommended values are accepted. Deviating national regulations were defined only in the case of seismic loading.

Fig. 1 shows schematically the temporal development of the various documents. In addition, design rules are also included in various approvals.

Figure 1: Temporal development of the different design guidelines
In the EOTA Working Group "Metal Anchors", it was stipulated that the design can be integrated into the relevant European Technical Approvals according to CEN / TS 1992-4 if the manufacturer so requests. As a result, the design of fasteners is currently possible both in accordance with ETAG 001, Annex C and EOTA TR’s as well as CEN / TS 1992-4, if a corresponding reference is included in the European Technical Product Specification and the necessary values are given. These include:

- Characteristic resistances for different failure modes,
- Factors for calculating the characteristic resistance (e.g. factor for calculation of concrete cone resistance)
- Geometrical data (e.g. embedment depth, characteristic edge distance and spacing)
- Minimum distances for the installation and minimum dimensions of the concrete member

The various above-mentioned, parallel applicable design rules are basically similar but not identical. This leads to confusion among the users. Furthermore, in the future European Technical Product Specifications within the meaning of the Construction Products Regulation (CPR), no regulations for dimensioning, but exclusively for product qualification, shall be included. Therefore, the compilation of the design rules in a single standard with a European scope is a significant step forward.

2 Essential elements of EN1992-4 and differences to the previous provisions

EN 1992-4 has been revised to include the current state of the art, and has been extended to include important special topics. The topics were summarized in the main part or in so-called CEN Technical Reports (CEN / TR). The following is a summary and explanation of the most significant changes.

a) Use of concrete cylinder strength instead of concrete cube strength in all design equations

This partly changes the factors and product-specific values necessary for the design. For this reason, only those European Technical Product Specifications may be used which explicitly refer to the dimensioning according to DIN EN 1992-4.

b) Consideration of the interaction between tension and shear loading differentiating the failure modes

According to the previous provisions, the minimum resistance from all failure modes under tension and shear load was used. This approach usually yields to very conservative results because different types of failure and the resulting forces are superimposed. In addition, the stresses can occur at different locations (e.g. concrete failure under tension load and steel failure under shear load). The improved approach in EN 1992 4 represent the actual behaviour better and follows other regulations (such as in steel construction) (see Fig. 2). This approach applies to all treated fastening systems (headed studs, anchor channels, dowels).
Figure 1: Combined tension and shear loading – comparison of design curves (acc. to Ref.\textsuperscript{14})

Curve 1 – previous model: use of minimum values from all failure modes
Curve 2 – improved model: accounting for different failure modes and superposition

c) Technical Report „Anchor channels“\textsuperscript{3}

While in the main part only anchor channels under tension and shear loads perpendicular to the longitudinal axis are treated, anchor channels with shear load in direction of the longitudinal axis can be calculated according to the Technical Report. However, this application is limited to channels with 2 or 3 anchors. The load introduction may only be carried out by means of screws, which ensure a mechanical interlock with the channel profile. The mechanical interlock can either be carried out using so-called notched screws in conjunction with normal channel profiles or by means of toothed screws in conjunction with tooth profiles (Fig. 3). A load introduction by friction alone is excluded.

In the Technical Report, a design model for consideration of supplementary reinforcement, which is improved compared to the main part, is also given. The model essentially differs from the approach in the main part, because an increased proportion of the load transferred through the hook of the rebar is applied. This is mainly due to the fact that the compression struts of the strut and tie model have a favorable effect on the load transfer of the reinforcing bar. Furthermore, in this model, the effectiveness of the supplementary reinforcement as a function of the distances of the rebars to the anchors and as a function of the position in the concrete break-out body is considered by factors. In addition, a distinction is made between the anchoring in cracked or uncracked concrete. In total, with this model is usually a much more economical dimensioning possible.
d) Technical Report „Plastic design“

In CEN / TS 1992-4, a plastic design concept was given for the calculation of the different anchor rows. This model is taken up and extended in the Technical Report. In addition, the models have been harmonized with the models included in EN 1993 and EN 1998.

e) Technical Report „Redundant systems“

CEN / TS 1992-4 did not consider the design of redundant systems (statically indeterminate multiple fixtures, such as suspended ceilings, façades, pipelines). In the Technical Report, the model was adopted in accordance with ETAG 001, as no design procedure would have been available in the future.

Redundant systems are defined by the fact that, in the event of a failure of an anchorage point, the loads can be transferred to the adjacent anchorage points. Two conditions must be met:

- The presence of a statically undetermined system in which the loads can be transferred to the adjacent anchorage points in such a way that they are not overloaded and failure occurs.
- The member to be fixed (anchor plate, pipeline, façade, suspended ceiling, etc.) must be designed in such a way that the loads can be transferred.

f) Design of post installed anchors under seismic action

Until now, a design method for the use of post installed anchors and cast-in headed studs for load-bearing connections or fastenings in earthquake areas was only available in the Technical Specification and the Technical Report. For applications in nuclear power plants, corresponding regulations for the consideration of earthquake excitations have existed for a long time. Although DIN EN 1998-1 contains special rules for composite structures made of steel and concrete in section 7, connections with dowels and headed studs are not covered. Special conditions must be observed here.

The current rules in EN 1992-4 distinguish between connections between structural elements or between non-structural attachments and structural elements. Two performance categories C1 and C2 are defined for certain earthquake loads. The corresponding product-specific parameters are described according to Ref. in the respective European Technical Product Specifications.

EN 1992-4 provides the current state of the art for the design of fastenings in concrete and enables an efficient and economical design. In addition, the introduction throughout Europe guarantees a uniform approach. Both aspects offer planners considerable advantages, both for national as well as cross-border tasks. The design concept is based on equally uniformly qualified products and thus offers a sufficient safety level.
3 DAfStb booklet No 615 – Commentary to EN 1992-4

The large product diversity and the steadily growing range of applications require more and more special knowledge for the planners and users in order to be able to optimally use such anchors. In particular, design and installation have become much more complex in the past few years, which means that this knowledge is often not given in a comprehensive way in the current education and training of the engineers. The information in the DAfStb booklet 615 are intended to facilitate the understanding and use of EN 1992 4 by explaining and presenting the scientific basis. The booklet 615 consists of three parts.

The first part of the booklet contains explanations of the standard text, notes on derivations of some regulations, supplementary application rules as well as some design examples. The second part explains the technical reports cited in the standard.

The first and second parts of the booklet are compiled in the Technical Subcommittee "Fastening Technique" of the DAfStb and approved in a standard-like process.

The third part of the booklet contains contributions written by members of the subcommittee "Fastening Technology" on its own responsibility and containing further explanations on selected topics. The contributions deal in particular with applications which are not covered in DIN EN 1992-4 and provide engineering solutions.

4 Summary

DIN EN 1992 4 has been extended to represent the current, generally recognized state of the art. These topics have been partially taken into account in the main part or summarized in so-called Technical Reports (TR).

The DAfStb had already decided during the processing of DIN EN 1992-4 to develop a commentary text for EN 1992-4. The information in the DAfStb booklet 615 are intended to facilitate the understanding and use of EN 1992 4 by explaining and presenting the scientific basis. In order to significantly increase the acceptance of the commentary book in the engineering community as well as outside Germany, the booklet is also published in English.

With the EN 1992-4 and DAfStb-booklet 615, an extensive tools will be available in future in order to design and dimension anchors in concrete safely and economically.

References:

   Teil 1: Allgemeines
   Teil 2: Kopfbolzen
   Teil 3: Ankerschienen
   Teil 4: Dübel – Mechanische Systeme
   Teil 5: Dübel – Chemische Systeme

3. CEN/TR 17080: Bemessung der Verankerung von Befestigungen in Beton – Ankerschienen – Ergänzende Regeln

4. CEN/TR 17081: Bemessung der Verankerung von Befestigungen in Beton – Plastische Bemessung von Befestigungen mit Dübel und Kopfbolzen

5. CEN/TR 17079: Bemessung der Verankerung von Befestigungen in Beton – Redundante nicht tragende Systeme


CRACK MOVEMENT TEST – DIFFERENCES IN EUROPEAN AND AMERICAN STANDARDS

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ABSTRACT

Over the years the so called crack movement test, respectively crack cycling test, developed in the different guidelines apart. This abstract shows the main differences in the specifications between Europe and the USA for the crack movement test and summarizes parameters influencing the reproducibility of the test results.

For the assessment of the product performance in cracked concrete the crack movement tests have to be performed according to ETAG 001 respectively EAD 330232-00-0601 in Europe or according to ACI 355 respectively AC 193/ AC 308 in the USA.

The main difference is the calculation of the permanent load \( N_p / N_{w,red} \), especially, if pullout gets decisive. According to ETAG 001 there are certain defined load levels that have to be taken into account for the calculation of the test loads. In the new EAD 330232-00-0601 these levels became closer.

In the EAD 330232-00-0601 the load has to be calculated with the characteristic value of the ETA, not anymore with reference values of tests.

In the American standards the proportion to the characteristic load of the reference test is decisive and there are no certain levels required if the load has to be reduced.

For the permanent load calculation according to ACI 355.2, the concrete strength of the test member is taken into account.

In the European Guidelines the concrete of the test member has to keep a certain compressive strength between 20 N/mm² and 30 N/mm² \( f_{c,eyl} \) respectively 25 N/mm² and 35 N/mm² \( f_{c,cube150} \) (C20/25), but with no influence on the calculation of the test load \( N_p \).

Also other parameters and frame conditions within the test procedure may be interpreted slightly different. Chapter 4 gives an overview over the not exactly defined test parameters and shows potential for improvement in order to increase the reproducibility of the test.
1 Introduction

In times of globalization it gets more and more important that a product should meet the requirements of different specifications all over the world. This paper shows how test series with originally the same purpose may develop differently over the years in the European and American specifications. The so-called crack movement test serves as an example. Currently manufacturers have to perform these test series twice with slightly different parameters to fulfill the specification, which causes additional work and costs.

In the following the main differences between the valid specifications for mechanical anchors in Europe according to EAD 330232-00-0601 and in the US according to ACI 355.02-07 respectively the AC 193 by ICC-ES are described in detail. The differences shown on the example of the mechanical fasteners are also valid for bonded anchors, in principle.

2 History

The crack movement test was implemented as a standard test for anchors used in cracked concrete with the introduction of EATG 001 in 1997. In the USA the test was implemented in 2002 in ACI 355.2-01 and in AC 193 „Acceptance Criteria for Mechanical Anchors in Concrete Elements“.

In 2006 the adhesive anchor codes were published and the test was described in the provisions AC 308 „Acceptance Criteria for Post-Installed Adhesive Anchors AC 308“ and ACI 355.04. These descriptions left room for interpretations. Therefore, in January 2012 a memo [9] was published by ICC-ES staff with clarification of language pertaining to the test for sensitivity to crack width cycling ("crack movement test") in AC193 and AC308.

In 2016 the European Assessment Document EAD 330232-00-0601 “Mechanical Fasteners for Use in Concrete” was published and is now the valid document for mechanical anchors in Europe. The test procedure is specified in EOTA Technical Report 048.

3. Description of the crack movement test

3.1 Purpose of the test

The purpose of the tests is the same in the American and European specifications. The crack movement test simulates the effect of crack opening and closing on the anchor tension resistance as it might occur over the anchor service life due to load changes, diurnal temperature changes, settlement, or restraint of shrinkage and creep. 20 load changings with influencing crack width per year are assumed. So for 50 years of service life, as specified in the specifications, 1000 crack movements are requested in the test.
3.2 Test member and test procedure according to EOTA TR 048

The tests are carried out on test members with unidirectional cracks. The crack width shall be approximately constant throughout the member thickness. The thickness of the test member shall be $h \geq 2 h_{ef}$ but at least 100 mm. To control cracking, so-called ‘crack-formers’ may be built into the member, provided they are not situated near the anchorage zone. An example for a test member is given in Figure 1:

In the test with variable crack width the reinforcement ratio (top and bottom reinforcement) of the concrete member shall be $\mu = A_s / (b \cdot h) \sim 0.01$ and the spacing of the reinforcing bars $\leq 250$ mm.

After installation of the fastener the maximum (max $N_s$) and minimum (min $N_s$) loads applied to the test member shall be determined such that the crack width at max $N_s$ is $\Delta w_1 = 0.3$ mm and at min $N_s$ is $\Delta w_2 = 0.1$ mm. To stabilize crack formation, up to 10 load changes varying between max $N_s$ and min $N_s$ may be applied while the fastener is installed but not loaded. Then a tensile load $N_p$ as specified in the relevant EAD is applied to the fastener after opening the crack to $\Delta w_1 = 0.3$ mm.

The load $N_p$ on the fastener shall remain constant during the test (variation $\pm 5\%$). Then the crack is opened and closed 1000 times (with a frequency of approximately 0.2 Hz). During opening of the cracks, the crack width $\Delta w_1$ is kept approximately constant (see Figure 2); for this purpose the load max $N_s$ applied to the test member may have to be reduced. The load min $N_s$ is kept constant. Therefore, the crack width $\Delta w_2$ is allowed to increase during the test (see Figure 2). The crack width difference $\Delta w_1 - \Delta w_2$, however, shall be $\geq 0.1$ mm during the 1000 movements of the crack. If this condition cannot be fulfilled with $\Delta w_1 = 0.3$ mm, then either min $N_s$ shall be reduced or $\Delta w_1$ be increased accordingly.

[Diagram showing test member and crack inducers]

Figure 1: Example of a test member for fasteners tested in cracked concrete according to TR 048

The load/displacement behavior shall be measured up to the load $N_p$. Afterwards under $N_p$, the displacements of the anchor and the crack widths $\Delta w_1$ and $\Delta w_2$ shall be measured either continuously or at least after 1, 2, 5, 10, 20, 50, 100, 200, 500 and 1000 crack movements.
After completion of the crack cycling the fastener shall be unloaded, the displacement measured and a tension test to failure performed with $\Delta w = 0.3$ mm.

The tests shall be performed for all diameter sizes of the fastener. The tests are performed in concrete C20/25.

The holes shall be drilled with a cutting diameter $d_{\text{cut,max}}$ (for torque-controlled and deformation-controlled fasteners) and diameter $d_{\text{cut,m}}$ (for concrete screws and undercut fasteners) of the drill bit according to EOTA Technical Report 048.

In ACI 355.2-07 the pictures are comparable; differences in the content will be discussed in the following chapters, but in general the description is quite comparable.

4 Significant differences between the American and European specifications

4.1 Calculation of the static tension load applied during the test

4.1.1 Calculation of the permanent tension load during the test according to ACI 355.2-07

$$N_w = 0.3N_{\text{p,cr}} \sqrt{\frac{f_{c,\text{en}}}{f'_c}}$$  \hspace{1cm} (8-1)

where

$N_w$ = static tension load applied to anchor during crack width cycling;

$N_{\text{p,cr}}$ = characteristic pullout resistance in cracked concrete for the minimum specified concrete strength of 2500 psi (17 MPa);

$f'_c$ = specified concrete compressive strength of 2500 psi (17 MPa);

$f_{c,\text{test}}$ = mean concrete compressive strength as measured at time of testing.
4.1.2 Calculation of the permanent tension load during the test according to EAD 330232-00-0601:

\[ N_p = 0.50 \frac{N_{Rk}}{\gamma_{inst}} \]

where

\[ N_{Rk} = \text{characteristic tensile resistance in cracked concrete C20/25 intended to be published in the ETA} \]

\[ \gamma_{inst} = \text{robustness factor (formerly designated as installation safety factor)} \]

If the displacement criteria (see Figure 3) is not fulfilled, the test has to be repeated with a lower load \( N_{p,\text{red}} \) and in the ETA the failure mode “pullout” has to be reported. For the reduced load the requirements of Table 1 have to be kept.

![Figure 3: Allowable displacements during crack movement test according to EAD 330232-00-0601](image)

<table>
<thead>
<tr>
<th>Range of ( N_{Rk,p} ) [kN]</th>
<th>Increment [kN]</th>
<th>Example [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \leq 10 )</td>
<td>0.5</td>
<td>3.0 / 3.5 / 4.0 … 9.5 / 10.0</td>
</tr>
<tr>
<td>( \leq 20 )</td>
<td>1.0</td>
<td>11.0 / 12.0 … 19.0 / 20.0</td>
</tr>
<tr>
<td>( \leq 50 )</td>
<td>2.0</td>
<td>22.0 / 24.0 … 48.0 / 50.0</td>
</tr>
<tr>
<td>( &gt; 50 )</td>
<td>5.0</td>
<td>55.0 / 60.0 / 65.0 / …</td>
</tr>
</tbody>
</table>
4.2 Difference in the test load level - example

In the following example for calculating the test load levels according to European and American specifications is presented:

Assumptions:

Mechanical Anchor, anchorage depth = 50 mm, pullout failure is decisive, $\gamma_{\text{inst}} = 1.0$.

Only varying parameter is the concrete strength for the 1st calculation with $f_{\text{c,cyl}} = 20 \text{ N/mm}^2$ the 2nd calculation $f_{\text{c,cyl}} = 30 \text{ N/mm}^2$, highest and lowest level according to TR 048.

According to EAD 330232-00-0601 the load $N_p$ is the same for both conditions:

$N_p = 0.50 \times N_{Rk}$

According to ACI 355.2-07 the load $N_{p,cr}$ with $f_{\text{c,cyl}} = 20 \text{ N/mm}^2$

$N_w = N_{p,cr} \times 0.3 \times (20/17)^{0.5} = 0.33 \times N_{p,cr}$

According to ACI 355.2-07 the load $N_{p,cr}$ with $f_{\text{c,cyl}} = 30 \text{ N/mm}^2$

$N_w = N_{p,cr} \times 0.3 \times (30/17)^{0.5} = 0.40 \times N_{p,cr}$

Assuming the same characteristic pull-out load $N_{Rk}$ of 10 kN at a concrete strength $f_{\text{c,cyl}} = 20 \text{ N/mm}^2$, the characteristic value of $N_{p,cr}$ according to ACI 355.2-7 has to be increased with $(20/17)^{0.5}$ (because of the different minimum value for the concrete strength between ACI 355.2-07 17N/mm² and EAD 330232-00-0601 20 N/mm²). This results in the following loads for the tests:

According to EAD 330232-00-0601 the load $N_p$ is the same: $N_p = 0.50 \times 10 \text{ kN} = 5 \text{ kN}$

According to ACI 355.2-07 $N_w$ with $f_{\text{c,cyl}} = 20 \text{ N/mm}^2$: $N_w = 10 \text{ kN} \times 0.33 \times (20/17)^{0.5} = 3.6 \text{ kN}$

According to ACI 355.2-07 $N_w$ with $f_{\text{c,cyl}} = 30 \text{ N/mm}^2$: $N_w = 10 \text{ kN} \times 0.40 \times (20/17)^{0.5} = 4.3 \text{ kN}$

So the test loads according to ACI 355.2-07 are only about 70% to 85% of the loads by EAD 330232-00-0601. In connection with the displacement criteria this could be the decisive factor for the tensile load bearing capacity of the anchor reported in the ETA respectively in the ICC-ESR.

5 Parameters influencing the reproducibility of crack movement tests

One of the most important objects of the specifications is the reproducibility of the test results to ensure a level playing field for the anchor manufacturers and to facilitate the comparability of the performance of different anchors for the designers. Every test series should be described and defined that every testing lab with a certain qualification and experience in testing anchors should be able to perform these tests and the results should be comparable (within a small variation) with the results of the same tests series performed in another qualified testing lab. The following sections show testing parameters with a certain range of variation which in summary have a significant influence on the test results. In combination with the absolute requirements within the crack movement test like the displacement criteria, these parameters could be decisive for the performance of the anchor.
5.1 Reinforcement
An important influence on the test may be the reinforcement of the test specimen serving as base material for the anchors. The reinforcement ratio \((\mu = A_s / (b \cdot h) \sim 0.01)\) is not exactly defined. Depending on the cross section of the concrete member, it is quite probably, that the reinforcement ratio cannot be met exactly. There is no information, weather it would be better to provide a reinforcement ration bigger or smaller than 0.01 and where would be the limit to be technically taken into account. The reinforcement size/diameter and spacing as well as the distribution of the reinforcement over the member depth is freely choose able. Also the pre-debonding length of the reinforcement in the area of the cracks could be defined individually by the testing lab.

5.2 Test specimen
An influence on the reproducibility of the test results may also have the member dimension. Normally the tests are performed with a member thickness larger than \(2 \text{ h}_{\text{ef}}\) in members with 200, 300 or 400 mm. The tests can be performed in both, either the formed or the troweled side. Even if the crack opening is provided by crack inducers supporting the opening of the cracks initiated by tensile loading on the reinforcing bars, praxis shows, the crack opening mechanism could be different in testing labs depending on the test equipment. At least one testing lab opens the cracks by expanding oil-filled tubes, which creates a completely different stress situation in the test member than found in practice and simulated by crack inducers and tension loaded reinforcing bars. Additionally, the provided range of the concrete strength allows a certain range of variation, too. This is valid also in the case where a normalization with respect to concrete strength is performed (see [1; 2, 3, 4]).

5.3 Control systems
The cycling frequency is defined smaller or equal 0.2 Hz and could be very slow. Also the loading and unloading rates and the servo control of the static load could have a certain influence. It is also possible to test more than one anchor in a crack simultaneously what certainly has an influence on the crack closing. And there are often test members with more than one crack in a row which are all opened and closed during each test. So probably the last crack of 5 cracks used has already experienced 4000 crack openings and closings before the test starts. Another topic is how and where to measure the crack with. Depending on the thickness of the member and the way of loading the reinforcement or inducing the cracks the crack width could vary over the member thickness (parallelism of the crack over the thickness of the test member).

Also the position of the crack crossing the drill hole can vary strongly. Even if crack inducers are used, the crack develops freely and does not always cross the drill hole as requested. Finally, it is not defined, how the crack should cross the drill hole, neither vertically nor horizontally. So it is possible, that the crack is not always crossing the entire zone, where the anchor transfers the load to the concrete. Some testing labs check that with endoscope cams but it’s not clearly described.
5.4 Test and Assessment

The difference in determination of the test load is already discussed in chapter 4. Another discussion during the last years is dealing with the control of the crack width. In the US, in January 2012 a memo [9] was published by ICC-ES staff with clarification of language pertaining to the test for sensitivity to crack width cycling in AC193 and AC308. Mainly in Europe different crack width protocols have been discussed from a constant $\Delta w = \Delta w_1 - \Delta w_2 \geq 0.1$ mm over the curve printed in the picture of the ETAG/EAD (see Figure 2) up to a linear increase on a logarithmic scale of the crack openings. One reason for that was the development of test equipment which is now able to steer the crack width. So more and more testing labs are able to run the test with a defined increasing crack from 0.1 mm at the beginning of the test to the limit of 2 mm at the end of the test. The discussion about steering the lower crack width is still running.

Figure 4: Crack opening variations during the crack movement test – discussion area

6 Conclusion

To increase the reproducibility of the crack movement tests and to get a defined level playing field for all kind of anchors regardless of where they have been tested, the test procedure and their description should be improved. There are several possibilities that may be changed.

It is not to understand why the drill bit diameter is defined differently for concrete screws and undercut anchors. Especially for concrete screws experience show, that for such fasteners the user prefers to use new drill bits to reduce the installation effort. However, a new drill bit usually has a cutting diameter which is bigger than $d_{cut,m}$. So the question has to be raised, why the crack movement test is different in this point for concrete screws and undercut anchors? Especially for concrete screws the cutting diameter in this test has a strong influence on the performance assessment. And additional characteristic of concrete screws is a very stiff behaviour like adhesive anchors. After a certain displacement limit they are completely by pulled out of the concrete.

A strictly definition of the concrete member would improve the reproducibility of the tests for sure. But there are a lot of different type of testing machines and equipment developed in the testing labs.
Such a change would cause a lot of investments and so the costs for testing may increase tremendously.

The biggest issue is that the crack width could not exactly be defined. Depending on the working principle of the anchor and the pre-stress force of the anchor the closing of the crack could be blocked and may not be closed to 0,1 mm during the 1000 cycles anymore. So probably the delta between $\Delta w_1$ and $\Delta w_2$ could vary between 0,1 mm and 0,2 mm. A possibility of improvement could be to fix a certain $\Delta w$ for example of 0,15 mm which has to be kept constantly during the test. With the current software and equipment this would be possible without a big invest. Disadvantage of that, would be, that lots of products had to be retested.

In the EAD 330232-00-0601 the concrete strength should be included in the calculation of the test load. So the differences concerning the test load level between the European and US specifications could be equalized.

In my opinion the best way of improvement would be to eliminate the pass–fail criteria for the displacement. Probably it may be possible to find a new evaluation of the displacements and to define certain limits. For me it is not reasonable why an anchor with 3,01 mm displacement after 1000 crack cycles should have a smaller pullout capacity than the other with 2,99 mm. I could imagine a kind of stiffness criteria on an evaluation of the characteristic or mean value of the displacements for example or to test different load levels and make an evaluation between the mean displacement values.

So there will be a lot of discussions how to improve these tests, but hopefully this summary of the facts in this paper will help to find a compromise and will improve the test procedure.

7 Acknowledgement

Finally I want to thank ACI 355 TG 4 “crack movement test” with chair John Silva as well as the participating and manufacturers and testing labs from all over Europe and the USA contributing with their experience and background information on the application of the codes and the performance of the tests.

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9. MEMO from ICC-ES staff, subject: “Clarification of language pertaining to the test for sensitivity to crack width cycling in AC 193 and AC 308”, January 31, 2012
NEW DESIGN APPROACHES FOR METAL INJECTION ANCHORS IN MASONRY

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ABSTRACT

Since the publication of the guideline ETAG 029 “Metal Injection Anchors for use in Masonry” in 2010 anchor manufacturers are enabled to apply for technical documents, ETA, that are valid and accepted on the entire European market. Thereto, in the past decade the load-bearing behavior of injection anchors in masonry has been investigated in several publications. The design proposals cover tension, shear and oblique loading of single and group fastenings in masonry made of solid and hollow or perforated bricks as well as influences of joints and edges.

However, the assessment of an anchor system is still mainly based on tests and the characteristic values given in the ETA are only valid for the brick types tested. In addition, because of the broad variety of masonry bricks available on the market, only a selection of representative brick types can be included in an ETA. For brick types that are not listed, but an appropriate brick of the same type and base material (e.g. hollow clay brick) is listed in the ETA, anchors must be qualified by additional tests carried out on construction works. Further, the design according to ETAG 029, e.g. for anchor groups or at edges, requires either many test series within the assessment process, or very conservative default values apply.

This paper presents a summary of the design proposals that are yet not included in the existing regulations, their characteristics and further open questions that are to be investigated to achieve a more comprehensive design model for injection anchors in masonry. As many publications are only available in German, it is meant to stipulate the further development of the regulations.

1 Introduction

Metal injection anchors are applied in masonry for almost 40 years. The first national German approval of an injection anchor system for masonry was issued in 1979. The first approval for a ready-to-use injection mortar system consisting of a two component hard-cartridge as they are commonly used nowadays was issued in 1986. A description of the different injections systems and brick types can be found in 10, 12 and 13.

Since the publication of the guideline ETAG 029 “Metal Injection Anchors for use in Masonry” in 2010 anchors can be uniformly assessed in Europe. It enabled anchor manufacturers to apply for European technical documents, ETA, that are valid and accepted on the entire European market. Due to
changes in EU legislation, at present a transition of the guideline into a European Assessment Document is in progress with the existing regulations being unchanged (see Section 2).

In the assessment, the determination of characteristic resistances and installation parameters depending on the base-ground masonry is mainly based on test results that are in the first instance only valid for the specific brick types tested (i.e. solid brick or perforated or hollow brick made of clay, calcium silicate, concrete and autoclaved aerated concrete). Overall the concept already requires a quite considerable number of tests per brick type to obtain an economical and safe anchor design. In addition, because of the broad variety of masonry bricks available on the market, only certain representative brick types are included in an ETA. For brick types that are not listed in an ETA, but an appropriate brick (e.g. hollow clay brick) is included in the ETA, anchors must be qualified by additional tension tests carried out on construction works. Further, in the design according to ETAG 029 some regulations are rather conservative or not satisfactory, e.g. the handling of anchor groups, especially at the edge under shear loading.

Concerning the state of knowledge, up to now only few of the existing design approaches are represented in ETAG 029\(^4\) respectively the new TR 054\(^9\). Indeed, in the past decade the load-bearing behavior of injection anchors in masonry has been investigated in several publications (e.g. \(^{11}\) to \(^{13}\)). The proposed design approaches consider tension, shear and oblique loading of single and group fastenings in masonry made of solid and hollow or perforated bricks with and without edge influences. In Hofmann, Schmieder, Welz\(^{14}\) and Hofmann, Welz\(^{15}\) design issues more related to practice are treated. As these and many other existing publications are only available in German language, this paper is meant to stipulate the further development of the next generation of regulations.

## 2 Assessment and Design of Injection Anchors in Masonry in Europe

After the endorsement of a revised edition of ETAG 029 in April 2013\(^4\), at present a transition of the guideline into a European Assessment Document and a corresponding EOTA technical reports is in progress (without technical changes) to be in accordance with the relevant provisions of the European Construction Products Regulation (EU) 305/2011. For better understanding the actual status of the transition of the different parts is specified in Table 1.

<table>
<thead>
<tr>
<th>Actual situation</th>
<th>After transition</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Number and issue date</strong></td>
<td><strong>Part</strong></td>
</tr>
<tr>
<td>ETAG 029 Apr. 2013(^4)</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Annex A</td>
</tr>
<tr>
<td></td>
<td>Annex B</td>
</tr>
</tbody>
</table>

As the technical aspects within the transition remain unchanged, it is assumed that they are known to the reader and a repetition is omitted herein. The required proofs for the design in the ultimate limit state (method A) according to TR 054\(^9\) are summarized in Table 2. The proofs, for which new design approaches are presented in this paper, are greyed out.
Table 2: Required proofs according to TR 054\(^9\) (notations and symbols see therein)

<table>
<thead>
<tr>
<th>Resistance to tension loads</th>
<th>Resistance to shear loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Failure of the metal part</td>
<td>Failure of the metal part, shear load with or without lever arm</td>
</tr>
<tr>
<td>(N_{\text{Ed}} \leq N_{Rk,s} / \gamma_{Ms})</td>
<td>(V_{\text{Ed}} \leq V_{Rk,s} / \gamma_{Ms})</td>
</tr>
<tr>
<td>Pull-out failure of the anchor</td>
<td>Local brick failure</td>
</tr>
<tr>
<td>(N_{\text{Ed}} \leq N_{Rk,p} / \gamma_{Ms})</td>
<td>(V_{\text{Ed}} \leq V_{Rk,b} / \gamma_{Mm})</td>
</tr>
<tr>
<td>Brick breakout failure</td>
<td>Brick edge failure</td>
</tr>
<tr>
<td>(N_{\text{Ed}} \leq N_{Rk,b} / \gamma_{Mm})</td>
<td>(V_{\text{Ed}} \leq V_{Rk,c} / \gamma_{Mm})</td>
</tr>
<tr>
<td>Pull out of one brick</td>
<td>Pulling out of one brick</td>
</tr>
<tr>
<td>(N_{\text{Ed}} \leq N_{Rk,gb} / \gamma_{Mm})</td>
<td>(V_{\text{Ed}} \leq V_{Rk,gb} / \gamma_{Mm})</td>
</tr>
<tr>
<td>Influence of joints</td>
<td>Influence of joints</td>
</tr>
<tr>
<td>(N_{\text{Ed}} \leq \alpha \cdot N_{Rk,b} / \gamma_{Mm})</td>
<td>Not relevant, if joints are completely filled with mortar. Elsewise, consideration as free edge, if not otherwise specified in ETA.</td>
</tr>
</tbody>
</table>

### 3 New design approaches beyond existing regulations

#### 3.1 General

Masonry used as base-ground for anchorages differs in many ways from members made of concrete. Apart from the inhomogeneity caused by the masonry structure using rows of single bricks set next to another and mortar layers to connect the bricks, the bricks itself have a much wider range of compressive strengths from around 1.6 MPa to more than 40 MPa. They may also feature voids or a web-like cross-section with holes. This leads to more complex approaches for the design of fastenings to masonry than applicable for concrete. However, by implementing the new approaches into computer-assisted programs the design should be feasible also for the designing engineer.

The load bearing behaviour of metal injection anchors in masonry under tension loading was extensively investigated by Meyer\(^12\) for tension loading and by Welz\(^13\) for shear and oblique loading. In the following, their main design proposals are summarized and commented.

#### 3.2 Tension loading

##### 3.2.1 Pull-through and pull-out failure of the anchor

Meyer\(^12\) proposes equations (1) to (3) for the calculation of the characteristic resistances for failure modes related to the injection mortar. Equation (1) applies for pull-through failure of the anchor rod out of the mortar whereas equation (2) is valid for pull-out failure of the complete anchor out of the borehole (including injection mortar and a potential sieve sleeve). Equation (3) only applies for hollow or perforated bricks, where the mortar cone within the borehole may be sheared off at the web of the brick.

\[
N_{Rk,1}^0 = \tau_{Rk,1} \cdot h_{\text{ef}} \cdot d_s \cdot \pi \quad \text{[N]} \quad \text{Pull-through failure of anchor rod} \quad (1)
\]

\[
N_{Rk,2}^0 = \tau_{Rk,2} \cdot h_{\text{ef}} \cdot d_b \cdot \pi \quad \text{[N]} \quad \text{Pull-out failure of complete anchor (including mortar)} \quad (2)
\]

\[
N_{Rk,3}^0 = \tau_{Rk,3} \cdot (h_{\text{ef}} - h_{\text{vob}}) \cdot d_b \cdot \pi \quad \text{[N]} \quad \text{Shear failure of mortar cone in hollow bricks} \quad (3)
\]

\[
\tau_{Rk,1} = \text{Characteristic bond strength for failure between anchor rod and mortar [N/mm²]; to be determined by tests}
\]

\[
h_{\text{ef}} = \text{Embedment depth of anchor [mm]}
\]
d_S = Diameter of anchor rod [mm]

\( \tau_{Rk,2} = \) Characteristic bond strength for failure between base ground and mortar [N/mm²]; to be determined by tests in solid bricks and normalized to brick compressive strength

h'_{ef} = Effective embedment depth [mm]: h_{ef} for solid bricks;

for hollow or perforated bricks see Table 3

d_B = Drill hole diameter [mm]

\( \tau_{Rk,M} = \) Characteristic shear strength of injection mortar [N/mm²]; to be determined by tests

h_{web} = Accumulated thickness of all webs along embedment depth h_{ef} of anchor [mm]

Table 3: Determination of effective embedment depth h'_{ef} for hollow or perforated bricks

<table>
<thead>
<tr>
<th>1</th>
<th>Bricks with large holes, i.e. d_hole &gt; d_B (e.g. KSL6-1.4-10DF)</th>
<th>2</th>
<th>Bricks with small holes, i.e. d_hole ( \leq d_B ) (e.g. Hlz12-0.9-16DF)</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1.png" alt="Diagram 1" /></td>
<td><img src="image2.png" alt="Diagram 2" /></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

a. Completely back-filled webs

h'_{ef} = \( \sum h_{web,i} \) i = 1,2,...n

with h_{web,n} \( \leq h_{ef} - h_{Se} - h_{R} - h_{h,UK} \)

b. Case a + pure bond areas

h'_{ef} = \( \sum h_{web,i} + h_{web,E} \) i = 1,2,...n

with h_{hole,i} \( \leq h_{ef} - h_{Se} - h_{R} \) and \( \sum h_{hole,i} \leq h_{ef} - \sum h_{web,E} \) i = 1,2,...n

3.2.2 Brick breakout failure

The failure mode “brick breakout” is influenced by the anchor system and the base ground. It occurs, if the tension resistance of the brick is exceeded. Thus, the calculation is depending on the base material. For the calculation, Meyer\textsuperscript{12} proposes equations (4) to (6).

\( N_{Rk,c}^{o} = 1.4 \cdot \beta_{b,net} \cdot h_{ef}^{1.5} \) [N] Calcium silicate bricks

(4)

\( N_{Rk,c}^{o} = 5.5 \cdot \beta_{b,net}^{0.3} \cdot \sqrt{\rho} \cdot h_{ef}^{1.5} \) [N] Lightweight concrete blocks

(5)

\( N_{Rk,c}^{o} = 11.4 \cdot \beta_{b,net}^{0.5} \cdot h_{ef}^{1.5} \) [N] Clay bricks

(6)

\( \rho = \) Net dry density of brick according to EN 772-4 [kg/m³]

\( \beta_{b,net} = \) Net-brick compressive strength [N/mm²]; evaluated with actual solid area of cross-section

h'_{ef} = Effective embedment depth [mm]: h_{ef} for solid bricks;

for hollow or perforated bricks see Table 3
3.2.3 Influence of joints and edges and consideration of anchorage groups

Meyer investigated single anchors and anchorage groups (2 and 4 anchors) at different setting positions towards free edges and joints filled and not filled with mortar.

For single anchors, horizontal joints filled with mortar showed no influence on the load bearing capacity, whereas for vertical joints not filled with mortar - alike for anchorages at free edges - the capacity decreases with decreasing edge distance. This effect is considered by using the ratio of the areas \( A_{c,N} / A_{c,N}^0 \) in equation (7) with joints not filled with mortar being regarded as free edges.

In principle, the same applies for anchorage groups. However, the influence of joints not filled with mortar is smaller than for single anchors. Here, anchorages over joints not filled with mortar are implicitly covered by equation (7) due to the consideration of the spacing \( s \) in the calculation of the actual projected outbreak area \( A_{c,N} \).

\[
N_{Rk} = N'^0_{Rk} \cdot \frac{A_{c,N}}{A_{c,N}^0} \cdot \psi_{g,N} \quad \text{[N]} \quad \tag{7}
\]

\[
N'^0_{Rk} = \text{Characteristic resistance of single anchor without edge influences} \quad \text{[N]}
\]

\[
A_{c,N}^0 = (2c_{cr,N})^2 = s_{cr,N}^2 \quad \text{[mm}^2\text{]} \quad \text{Idealized projected area of brick breakout cone at the surface}
\]

\[
A_{c,N} = \text{Actual area of brick breakout cone at the surface. It is limited by overlapping breakout cones of adjoining anchors (} s \leq s_{cr,N} \text{) as well as free edges (anchor groups and single anchors) or joints not filled with mortar (only single anchors) of the masonry (} c \leq c_{cr,N} \text{).}
\]

\[
c_{cr,N} = \text{Required characteristic edge distance} \quad \text{[mm]}
\]

\[
c_{cr,N} = 1,5 \cdot h'_{ef} \quad \text{for brick breakout failure (see equations (4) to (6))}
\]

\[
10 \cdot d_s \cdot \left( \frac{\tau_{Rk}}{10} \right)^{2/3} \quad \text{for pull-out failure (} \tau_{Rk} \text{ see equations (1) to (3))}
\]

\[
s_{cr,N} = 2c_{cr,N} \quad \text{Required characteristic spacing} \quad \text{[mm]}
\]

\[
ds = \text{Nominal diameter of anchor rod} \quad \text{[mm]}
\]

\[
\tau_{Rk} = \text{See equations (1) to (3)}
\]

\[
\psi_{g,N} = \text{Factor to consider the positive effect in case of groups with narrow spacing.}
\]

\[
\psi_{g,N} = \psi_{g,N}^0 + (1 - \psi_{g,N}^0) \cdot \left( \frac{s}{s_{cr,N}} \right) \geq 1,0 \quad \text{[-]} \quad \tag{10}
\]

\[
\psi_{g,N}^0 = n^\alpha \quad \text{with} \quad \alpha = 0,7 \cdot (1 - \frac{\tau_{Rk}}{\tau_{Rk,max}}) \leq 0,5 \quad \text{[-]}
\]

\[
n = \text{Number of anchors in a group}
\]

\[
\tau_{Rk,max} = \text{Required bond strength for the formation of a full breakout cone}
\]

\[
\tau_{Rk,max} = 1,4 \cdot \frac{h_{df}^{0,5}}{h_{df}^{0,5}} \quad \text{[N/mm}^2\text{]} \quad \text{Calcium silicate bricks}
\]

\[
\tau_{Rk,max} = 5,5 \cdot \frac{P_{df}^{0,5} \cdot h_{df}^{0,5} \cdot h_{df}^{0,5}}{\pi \cdot d_{B}} \quad \text{[N/mm}^2\text{]} \quad \text{Lightweight concrete blocks}
\]

\[
\]

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The consideration of a positive group effect by the factor $\psi_{g,N}$ according to equation (10) is an adoption of an early approach used for the design of bonded anchors in concrete in a CEN-draft document\textsuperscript{6}. In the opinion of the author, the consideration of a positive group effect for applications in masonry is not constructive, as for groups other effects are more pronounced.

For anchors set in masonry, the setting position within the brick (especially for perforated bricks) or towards joints leads to differences in the load-displacement behaviour of single anchors. For anchor groups under tension loading this causes non-uniform load distribution and thus lower group resistances than predicted, if only the ratio of the areas $A_{t,N}/A_{o,N}$ are considered, especially for cases with large spacing ($s \geq s_{cr,N}$). Therefore, a group factor $\psi_{g,N}$ shall rather take account of the described adverse effect. The author proposes to apply a factor $\psi_{g,N}$ according to (14). Note that in Hofmann, Welz\textsuperscript{15} further simplified approaches for the calculation of the group tension resistance are presented.

$$\psi_{g,N} = \begin{cases} 1,0 & \text{for single anchors} \\ n^{-0.2} & \text{for anchorage groups} \end{cases}$$

### 3.3 Shear loading

Because of lower compressive strength of many brick types compared to concrete and the potential brick perforation by voids and holes, for fasteners under shear loading in masonry structures additional failure modes to steel rupture and pry-out failure must be considered.

#### 3.3.1 Design model for local brick failure

The investigations by Welz\textsuperscript{13} have shown, that for anchorages without edge influence a local failure of the brick in front of the anchor in the direction of the shear load is the predominant failure mode. Thereby, the base material gets compressed and plasticizes to a certain degree. Depending on the brick type (solid or perforated) and brick compressive strength, the restraint of the fixture (rigid or pin-joint) and the parameters of the anchor system (diameters of anchor rod and borehole, use of sleeve and sleeve material, embedment depth) up to two plastic hinges may develop in the anchor in combination with the local crushing of the brick. The proposed design model is a modification and enhancement of the model used for timber connections in e.g. Eurocode 5\textsuperscript{7} considering the relevant parameters for the injection system and the masonry. Thus, in a further step, it may be combined with the models used in Eurocode 5\textsuperscript{7}. In the following at first the model for solid masonry is introduced then the model for perforated masonry is further specified. For the covered scope concerning anchor and brick types see Welz\textsuperscript{13}.

**Design cases for solid masonry**

The design approach for solid masonry distinguishes four different cases (A to D - see Figure 1). Firstly, the degree of restraint of the fixture is considered by means of its thickness and the diameter of the clearance hole. The latter is negligible, if the specifications concerning the maximum diameter

$$\tau_{fR,\text{max}} = 11.4 \cdot \frac{\beta_b^{0.5}}{\pi \cdot d_b} \quad [\text{N/mm}^2] \quad \text{Clay bricks} \quad (13)$$

$$\beta_b = \text{Brick compressive strength} \quad [\text{N/mm}^2] \quad \text{evaluated according EN 772-1 (with outer dimensions of cross-section)}$$
of the clearance hole given in ETAG 029 and respectively EAD 330076 are kept. For thin fixtures (cases A and B) a pin-joint and for thick fixtures (cases C and D) a rigid restraint for the fastener are assumed. A thin fixture is given for a thickness of the fixture that is smaller than half the anchor rod diameter and a thick fixture for thicknesses equal or larger than the anchor rod diameter.

For thin fixture \( t_{\text{fix}} \leq 0.5d_a \):

\[
V_{Rk,b,A}^0 = 0.75 \cdot d_{\text{nom}} \cdot f_i \cdot h_{eq} \left( \sqrt{2} - 1 \right) \quad \text{[N]} \quad \text{Case A} \quad (15)
\]

\[
V_{Rk,b,b}^0 = 0.75 \sqrt{2 \cdot M_{Pl,H} \cdot d_{\text{nom}} \cdot f_i} \quad \text{[N]} \quad \text{Case B} \quad (16)
\]

For thick fixture \( t_{\text{fix}} > d_a \):

\[
V_{Rk,b,C}^0 = 0.75 \cdot d_{\text{nom}} \cdot f_i \cdot h_{eq} \left( 2 + \frac{4 \cdot M_{Pl,S}}{d_{\text{nom}} \cdot f_i \cdot h_{eq}^2} - 1 \right) \quad \text{[N]} \quad \text{Case C} \quad (17)
\]

\[
V_{Rk,b,D}^0 = 0.75 \sqrt{2(M_{Pl,S} + M_{Pl,H}) \cdot d_{\text{nom}} \cdot f_i} \quad \text{[N]} \quad \text{Case D} \quad (18)
\]
With

\[ d_{\text{nom}} = \text{Borehole diameter} \ [\text{mm}] \]
\[ h_{\text{ef}} = \text{Effective embedment depth} \ [\text{mm}] \]
\[ f_1 = \alpha_{\text{local}} \cdot \beta_b \quad \text{Local brick compressive strength} \ [\text{N/mm}^2] \]
\[ \alpha_{\text{local}} = \text{Increasing factor to take account of the local compressive strength; to be determined by tests (see e.g. Welz}^{13} \text{. For solid bricks } \alpha_{\text{local}} \geq 2.0; \text{ for hollow bricks } \alpha_{\text{local}} \geq 1.0) \]
\[ \beta_b = \text{Brick compressive strength} \ [\text{N/mm}^2]; \text{ evaluated according EN 772-1} \]
\[ M_{\text{PLS}} = d_{\text{s}}^3 \cdot f_{y,k} / 6 \quad \text{Plastic flexural resistance of threaded rod} \ [\text{Nmm}] \]  \hspace{1cm} (19)
\[ d_{\text{s}} = \text{Diameter of stress area of rod} \ [\text{mm}] \]
\[ f_{y,k} = \text{Steel yield strength} \ [\text{N/mm}^2] \]
\[ M_{\text{PLH}} = \alpha_H \cdot M_{\text{PLS}} \quad \text{Plastic flexural resistance of the composite anchor system} \ [\text{Nmm}] \]  \hspace{1cm} (20)
\[ \alpha_H = \text{Modification factor for plastic flexural resistance of the composite anchor system; to be determined by tests (see Welz}^{13} \text{: } \alpha_H \geq 1.0) \]

As one can see, the equations for cases C (equations (17)) and D (equation (18)) can be transferred into the cases A (equation (15)) and B (equation (16)), if the plastic flexural resistance of the threaded rod is neglected \((M_{\text{PLS}} = 0)\).

**Design cases for hollow or perforated masonry**

In principle, the design procedure for perforated masonry is analogue to solid masonry by differentiation of the fixture thickness. However, for the design cases the configuration of holes in the bricks must be considered. If an anchor only penetrates the outer web, the cases for solid masonry apply under consideration that instead of \(h_{\text{ef}}\) the thickness \(h_1\) of the outer web has to be inserted. In cases, where anchors also penetrate the first inner web, the calculation has to be modified. Thereto, the following equations consider anchorages in up to two webs of a brick. The equations may be used for the majority of anchorages in brick types on the market. For deeper anchorages that penetrate more than 2 webs the equations deliver conservative results, if the embedment depth is limited to the depth of the first inner web. Brick types that feature very slender web configurations, e.g. latest versions of light hollow clay bricks, may be calculated as solid bricks with a smeared and reduced local brick compressive strength.

To simplify the representation in this paper, in the following only the equations for a thick fixture are presented. As explained above for solid bricks, the cases A and B for a thin fixture can be derived from the cases C and D, if the plastic flexural resistance of the threaded rod is neglected \((M_{\text{PLS}} = 0)\). For a thick fixture the three possible distributions of the reaction forces for each of the cases C and D are displayed as C1 to C3 and D1 to D3 in Figure 2. The cases C3 and D3 are only possible, if \(h_1 \geq h_2\) applies. Like in solid material, the lowest values of all cases C and D gives the resistance of the fastener.

The equations for cases D2 and D3 comply with case D for solid material (see equation (18)). For anchorages in more than one inner web corresponding approaches are necessary, that are strongly dependent on the configuration of the web. For the prevailing geometries of bricks and the dimen-
sions of fasteners the thicknesses of the outer web and void are most decisive and therefore anchor-
gages penetrating more than one inner web may be led back to the described cases.

Figure 2: Design cases for thick fixture in perforated masonry

The characteristic resistances for hollow or perforated masonry in case of a thick fixture ($t_{\text{fix}} > d_s$) are calculated as follows:

$$V^0_{Rb,b,c1} = V^0_{Rb,b,c2} = 0.75 \cdot d_{\text{nom}} \cdot f_1 \left(2(h_1 + h_2)^2 + 4 \left(2h_1 + h_2 + h_L\right) \cdot h_L + \frac{M_{PL,S}}{d_{\text{nom}} \cdot f_1} - (h_g + h_L)\right)[N] \quad \text{Cases C1 and C2}$$

$$V^0_{Rb,b,c3} = 0.75 \cdot d_{\text{nom}} \cdot f_1 \left[2(h_1 + h_2)^2 + 4 \left(h_2 \cdot h_L + \frac{M_{PL,S}}{d_{\text{nom}} \cdot f_1}\right) - (h_1 + h_2)\right][N] \quad \text{Case C3 (only, if } h_1 \geq h_2)$$

$$V^0_{Rb,b,D1} = 0.75 \cdot d_{\text{nom}} \cdot f_1 \left[2h_1 \cdot h_L + \frac{M_{PL,S} + M_{PL,H}}{d_{\text{nom}} \cdot f_1} - h_L\right][N] \quad \text{Case D1}$$

$$V^0_{Rb,b,D2} = V^0_{Rb,b,D3} = \text{See equation (18) for case D in solid material} \quad \text{Cases D2 and D3}$$

With

- $h_1 = \text{thickness of outer web } [\text{mm}]$
- $h_L = \text{diameter/ depth of outer hole of the brick } [\text{mm}]$
- $h_2 = \text{thickness of first inner web } [\text{mm}]$

### 3.3.2 Local brick failure for anchorage groups without edge influence

According to Welz\textsuperscript{13}, for all types of solid bricks and hollow bricks (units) out of calcium silicate, lightweight aggregate concrete or clay according to national German codes (DIN V 106, DIN V 18151-100 and DIN 105-100) equation (24) may be applied for local brick failure of anchorage groups without edge influence. For other hollow or perforated brick types according EN 771-1 per fixing point only the resistance of a single anchor may be assumed, if no tests with groups are available, and the corresponding characteristic spacing in a direction shall be assumed as the brick dimension in the same direction (either height or length).
\[ V_{Rk,b} = V_{Rc,b}^{o} \left( 1 + \frac{s_V}{s_{cr,V,b}} \right) \left( 1 + \frac{s_H}{s_{cr,H,b}} \right) \leq n \cdot V_{Rc,b}^{o} \ [N] \] (24)

With
\[ V_{Rc,b}^{o} = \text{Characteristic resistance of single anchor according to section 3.3.1} \]

\[ s_V, s_H: \text{Existing spacing in vertical (} s_V \text{) and horizontal (} s_H \text{) direction (if both exist – i.e. group of 4 anchors) with } s \geq s_{\text{min}} = \max(3d_{\text{nom}} \geq 50 \text{ mm}) \text{ [mm]} \]

\[ n = \text{Number of anchors} \]

\[ s_{cr,V,b} = 125 \text{ mm} \text{ characteristic spacing for local brick failure (brick types see text)} \]

### 3.3.3 Brick edge failure for anchorages in solid bricks

For a single anchor at the edge of a solid member not influenced by the member thickness or further edges the characteristic resistance is given by equation (25). However, in masonry the formation of the breakout cone in case of brick edge failure may be limited by the dimensions of the brick (height and width). Especially under loading towards the free edge, the brick height may limit the breakout cone, if the load transfer via the mortar layer in the horizontal joint to adjacent bricks is not ensured, e.g. because of poor mortar properties or insufficient vertical compression stresses.

Therefore, for single anchors and anchor groups, in which all anchors are placed in the same brick at the free edge, this effect is considered by using the ratio of the projected areas \( A_{V,c} / A_{V,c}^{o} \) as given by equation (26) (see Figure 3). Note that for narrow thin members equation (26) would predict that the breakout failure load decreases as the edge distance \( c_1 \) increases. Therefore, analogue to the proposal given in Eligehausen/ Mallée/ Silva\(^{16}\), in the calculation a maximum edge \( c_1^* \) is defined as the larger value of \( a/3 \) and \( h/1.5 \) in equations (25) and (26). For group anchorages with loading towards the free edge always the anchors nearest to the edge become decisive for \( c_1 \), unless special measures ensure load transfer to the back anchors. For group anchorages with loading parallel to the free edge the balance point of the group may be taken as \( c_1 \).

![Figure 3: Single anchor under shear loading towards free edge of solid masonry](image-url)
Characteristic edge failure for single anchor
(set in brick closest to free edge)

\[ V_{Rk,c}^e = k \cdot \left( \frac{h_y}{d_{nom}} \right)^{0.2} \cdot \sqrt{d_{nom}} \cdot \sqrt{\beta \cdot c_{t,5}} \quad [N] \]

Characteristic edge failure for single anchor
and anchor groups under shear load towards
the edge

\[ V_{Rk,c} = V_{Rk,c}^e \cdot \frac{A_{c,V}}{A_{o,a}} \quad [N] \]

with

\[ k = \begin{cases} 
0.25 & \text{if load direction is towards the free edge.} \\
0.45 & \text{if load direction is parallel to the free edge.} 
\end{cases} \]

\[ d_{nom}, \beta_{St}, \text{and } s_V: \text{see sections 3.3.1 and 3.3.2} \]

\[ c_t = \text{applicable smallest edge distance to free edge} \]

\[ c_1 \leq c_{t,*} = \max(a/3; h/1.5) \quad \text{maximum applicable edge distance in case of loading towards the} \]

\[ \text{free edge; to be applied in equations (25) and (26) and in the calculation of } A_{0,c,V} \]

\[ a = \text{Height of single brick} \]

\[ b = \text{Width of single brick} \]

\[ A_{0,c,V} = 4.5c_1^2 \quad [\text{mm}^2] \quad \text{projected area of the fully developed breakout surface of a single anchor;} \]

\[ \text{idealized as a half- pyramid with base lengths } 1.5c_1 \text{ and } 3c_1 (\text{compare Figure 3}) \]

\[ A_{c,V} = \text{projected area of the present breakout surface (limited by height and width of the brick)} \]

\[ A_{c,V} = \min(3c_1 + s_V; 1.5c_1 + c_{2,1} + s_V; a) \cdot \min(1.5c_1; h) \quad [\text{mm}^2] \]

\[ c_{2,1}; c_{2,2}: \text{ Distances to the edges orthogonal to } c_1 \text{ with } c_{2,1} \leq c_{2,2} \quad [\text{mm}] \]

If a single anchor or some or all anchors of a group are installed in the second brick away from the
free edge equation (26) yields conservative results. The same applies for anchor groups that are
placed at the edge in two different bricks belonging to different rows of bricks. However, the potential
depends on the brick dimensions and material properties. Thus, to exploit this potential, additional investigations are required for the determination of safe resistances.

4 Summary and outlook

In this paper, new design approaches for injection anchors in masonry for the relevant failure modes
under tension (pull-out failure of the anchor and brick breakout failure; including influences of joints) and shear loading (local brick failure; brick edge failure) for single anchors and anchor groups were presented that are yet not included in existing regulations such as ETAG 029\(^4\) or TR 054\(^9\). Consideration of the findings in the regulations will improve the design of anchorages and help exploiting the potential of injection anchors while assuring a safe design. Furthermore, expensive and time-consuming tests may be reduced or omitted.

The presented approaches are based on tests carried out on selections of bricks. Meanwhile several ETAs for injection anchors including different brick types are on the market. For further validation and improvement of the proposed approaches an evaluation of the underlying assessment test data would be helpful. For applications under shear loading with edge influence, especially also for hollow bricks, further improvements are desirable. Also, no investigations with eccentric loading are available. In Meyer\(^{12}\) and by Welz\(^{13}\) further proposals for continuing works and open questions are presented.
References


DESIGN OF COLUMN BASES IN EN 1993-1-8:2020

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ABSTRACT

The paper summarises the improvements prepared for the design of column bases in the new version of Eurocode 3 part 1-8 for design of joints in steel structures (EN 1993-1-8:2020). Based on systematic review in 2015 restructuring of the document has been prepared. In the new version are summarised the principles of design of connections in Chapter 6 and the detailed description of design of joints is transferred to normative Annexes, A Structural properties of basic components, B Application rules for moment-resisting beam-to-column joints and splices connecting H or I sections, C Application rules for simple connections, and D Application rules for column bases. In this Annex is harmonised the friction coefficient between steel and concrete with best European practice. The text and tables are here simplified and the effect of prying forces is explained.

EN 1993-1-8:2020 offers analytical models which assure good prediction for most basic structural solutions. Analytical models of resistance are equipped by prediction of stiffness, which is called Component method (CM). Structural steel Eurocodes are also focused on the next step of structural design applying the Finite Element Analyses (FEA). In joints may FEA design be applied for distribution of internal forces in joint and the connectors may be analysed as components, which is called the Component based finite element method (CBFEM). In contrary to common standardised joints the assembly of internal forces may be done by engineering judgment and the FEA may predict the component behaviour. An example of application of CM and CBFEM is prepared to show the pros and cons of both methods based on tests performed in the laboratory of Brno University of Technology.

1 Introduction

The Second generation of the structural design codes Eurocodes is under preparation since 2015, see Figure 1 from Ref.¹. The technical work started with systematic review of standards for design of elements EN 1993-1-1:2005, connections EN 1993-1-8:2006², high strength steel EN 1993-1-12:2006, and structures exposed to fire EN 1993-1-2:2005. In the second phase were reviewed standards for thin walled EN 1993-1-3, plated EN 1993-1-5, shell EN 1993-1-6, and plated structures subject to out of plane loading EN 1993-1-7, see Figure 2 from Ref.¹. For steel structures were in 2015 established two project teams SC3.1 Design of sections and members according to EN 1993-1-1 and SC3.2 Joints and Connections according to EN 1993-1-8, which is working under convenor Milan Veljkovic, TU Delft, in composition Ana Girao-Coelho, SCI London, Primož Može, TU
The contract of project team (PT) is focussed on reduction in number of national choices, enhanced ease of use, stainless steel bolted connections, extension of design rules for welded joints and for mixed steels connections, further development of the component models for joints in order to increase the robustness of joints, integration of design rules for advanced material, hollow section joints, clarification and specification of the application of EN 1090-2, component model for column base plates, and fire safety of joints. PT’s are asked to improve clarity and understandability of technical provisions, accessibility to technical provisions and ease of navigation between them, consistency within and between the Eurocodes, by including state-of-the-art material the use of which is based on commonly accepted results of research and has been validated through sufficient practical experience and considering the second generation of the Eurocodes as an evolution avoiding fundamental changes to the approach to design and to the structure of the Eurocodes unless adequately justified. At the end of April 2017, PT SC3.2 prepared the second draft of EN1993-1-8:2020.

Experimental evidence and curve fitting procedures were and still are used for safe and economical design of steel connections. Resistance of connections is predicted based on analytical models of welds, bolts, and plates, as the estimated lever arm of internal forces. Zoetemeijer was the first who equipped this model with estimation of stiffness. The elastic stiffness was improved by the work of Steenhuis. Basic description of components behaviour in major structural steel connections was prepared by Jaspart for beam to column connections and by Wald et al. for column bases. The idea was generalised by Da Silva for 3D behaviour including nonlinear behaviour. Procedure starts with decomposition of a joint into components followed by their description in terms of normal/shear force deformation behaviour. After that, components are grouped to examine joint moment-rotational
behaviour and classification/representation in a spring/shear model and application in global analyses. Advantage of this CM is integration of current experimental and analytical knowledge of connections components behaviour – bolts, welds and plates. This provides accurate prediction of behaviour in elastic and ultimate level of loading. Verification of the model is possible using simplified calculation. Disadvantage of CM is that experimental evaluation of internal forces distribution is available only for limited number of the open section joint configurations. In temporary scientific papers, description of atypical components is either not present or has low validity and description of background materials. The CM is not developed for hand calculation but as a method for preparation of design tables or software tools.

Figure 2: Set of the structural steel design codes EN 1993-x-y

2 Component method

The design of column bases is provided in EN1993-1-8\textsuperscript{2,8} according to CM. The analytical models of resistances and stiffness are provided for components: concrete and base plate under compression, concrete components for tension, base plate in bending under tension, anchor bolt in tension, anchor bolts in shear and concrete components for shear. The component concrete and base plate under compression is studied based on prediction of design resistance of equivalent T-stub in compression and of its stiffness. The design of T-stub is prepared to limit the prying forces in the anchor bolts to select proper failure mode only based on contact of base plate to concrete block.

In EN1993-1-8:2020\textsuperscript{8}, the resistances of the anchoring components in tension for steel-to-concrete joints are calculated according to EN 1992-4:2017\textsuperscript{9}. The verification of tensile components is done for the ultimate limit state for the following failure modes: concrete cone failure, pull-out failure, concrete splitting failure, concrete blow-out failure, steel failure of reinforcement and anchorage
failure of reinforcement. Increase in the load-carrying capacity of the anchoring components may be reached, if the contribution of concrete components and the supplementary reinforcement next to the fasteners is considered simultaneously. The load transfer due to the reinforcement and the concrete may be determined by an appropriate stiffness model for the anchoring components. Further failure modes such as concrete breakout between the supplementary reinforcement should be considered, when the contribution of concrete and reinforcement is assumed.

The new version of the code don’t modify in deep the way of transfer of shear forces between the base plate and the support. It is also possible to use friction between concrete and steel, shear of anchor bolts and block shear connectors. The coefficient of friction between base plate and concrete is increase from 0.2 to 0.3 based on experimental evidences and national practice.

The resistances of the anchoring components in shear for steel-to-concrete joints may be calculated according to EN 1992-4:2017. The verification of shear components may be done for the ultimate limit state for the following failure modes: concrete pry-out failure, concrete edge failure, steel failure of supplementary reinforcement and anchorage failure of supplementary reinforcement.

The text of the standard, and namely for column bases, was rather simplified for ease of use and fast understanding. The example of simplification of all models by taking into consideration symmetrical joints is shown in Figures 3 and 4 for the determination of the lever arm, which rather simplified all expressions.

The design moment resistance $M_{j,Rd}$ of a column base plate subject to combined axial force and bending moment should be determined from equilibrium of internal forces based on the design tension resistance of one side of the joint $F_{T,Rd}$ and the design compressive resistance of the other side of the joint $F_{C,Rd}$. The initial rotational stiffness $S_{j,ini}$ of a column base plate subject to combined axial force and bending moment is calculated from the tension stiffness coefficient of one side of the joint $k_T$ and the compression stiffness coefficient of the other side of the joint $k_C$ for proportional loading.
Component based finite element method

3.1 Principle

The component based finite element method (CBFEM) uses finite element model and springs to determine the forces and checks the resistance in components. The steel plates are modelled using shell elements, the anchor bolt using nonlinear spring and the concrete pad as 2D contact elements of Winkler subsoil model. Geometrically linear and materially nonlinear analysis is used for stress-strain calculation. The plastic design of fastenings allows for nonlinear load distribution in anchors. The CBFEM focuses on generally loaded complex connections. However, the CBFEM, as any finite elements analysis, should be validated by experiments, verified by currently used design methods and equipped by benchmark cases, see Ref. for checking of the range of application and implementation.

3.2 Anchor bolt

The anchor bolt is modelled as a nonlinear spring with bilinear load-deformation diagram. The bolt is on one side fixed to the concrete block. Its length $L_b$ is taken according to EN1993-1-8:2006 as sum of half the thickness of a nut, washer thickness $t_w$, base plate thickness $t_{bp}$, grout thickness $t_g$ and free length embedded in concrete, which is expected as $8d$, where $d$ is bolt diameter. The stiffness in
tension is calculated as \( k = \frac{E A_s}{L_b} \). The nominal values of anchor bolt material are used in the load-deformation diagram of the nonlinear spring, see Figure 5. The spring model is dependent on yield strength, ultimate strength, limit elongation and modulus of elasticity. The steel failure is considered as ductile and failures concerning concrete, e.g. concrete cone failure \( N_{c,Rd} \), are considered as brittle and can occur at the elastic branch of the bilinear spring model. The prying forces are evaluated by finite element model and the resulting forces include the prying forces.

3.3 Concrete block

The concrete block is simplified as 2D contact elements due to the requirement for fast calculation. The contact is active only in compression and it uses the penalty method to determine the distribution of contact force. The stiffness of concrete in compression by Winkler subsoil model was determined by comparing the peak compressive stresses in advanced 3D models in Midas FEA\textsuperscript{13} and ATENA\textsuperscript{14} software. However, the resistance of concrete in compression is assessed using the plastic distribution as in CM in EN 1993-1-8:2006\textsuperscript{2}. The design bearing strength of concrete \( f_{jd} \) is checked against the average stress on the effective area \( A_{eff} \) under the base plate. This effective area is the intersection of two areas, the effective one determined by methods in EN 1993-1-8:2006 for compression only and the one in contact with subsoil determined by finite element analysis.

This approach ensures the assessment for general loading of any base plate. In the Figure 6, the dashed area is the effective area determined for compression only, the coloured area is the area in contact with subsoil, and the hatched area is the final effective area \( A_{eff} \).

3.4 Validation

The experimental program consisted of two column bases loaded by uniaxial moment and two column bases loaded by biaxial moment. All four column bases comprised HEB 240 columns 2 m long, base plate 330 \( \times \) 440 \( \times \) 20 mm, shear lug, and four headed cast-in anchors made of threaded...
rods M20. The steel was grade S235, the anchor bolts grade 8.8 and concrete grade C16/20. The embedment depth of anchor rods was 250 mm. The experiments are thoroughly described in Ref. 15 and the set-ups of column bases, 1, 2 is uniaxial loading and 3, 4 biaxial loading, are in Figure 7. All columns were loaded with nearly constant vertical load $F_v = 400$ kN and increasing horizontal force $F_h$.

The CBFEM has the same dimensions as the experiment but note that nominal values of material properties were used as it is a design method and not research tool, whose results should correspond completely with the experiment. The selected and verified mesh can be seen in Figure 6.

The comparison of CBFEM and the experiments is shown in Figure 8. The graph of uniaxial bending shows the moment rotation diagram around $y$ axis. In this case, the elastic deformation of the column was subtracted because the horizontal force and deformation was measured at the hydraulic cylinder inducing the horizontal force $F_h$. In the case of biaxial bending, the moment $M = 1.83 \times F_h$. $\delta_h$ is deformation in the direction of this horizontal force at its position. The accuracy of determination of the bending moment and rotation in the direction of principal axes $y$ and $z$ is limited due to decreasing stiffness varying in both directions with increasing horizontal force $F_h$.

The diagrams coincide closely and considering the nominal values used in the CBFEM, the method yields reasonably safe results. The stiffness is lower in the case of all experiments probably also due to the added deformations of the frame, to which the hydraulic cylinder was attached, the stabilizing beams, and the strong floor.

![Figure 7: Experiment set-ups](image-url)
The forces in anchors are essential for the determination of resistance of the column base. The comparison of forces in anchors determined by finite element analysis to experimentally measured ones in washers and strain gauges glued to the anchor rods is presented in Figure 9. The results show good agreement. The difference in the case of biaxial bending is caused by the earlier yielding of the base plate. The steel used in experiment had higher yield strength than nominal values used in CBFEM model, so the force in anchor rod B3 decreases sooner and at the same time force in B4 rises.

According to EN 1993-1-8:2006 and draft of EN 1992-4:2017, the following components are checked: column flange and web in compression, concrete in compression including grout, base plate in bending under compression, base plate in bending under tension, anchor bolts in tension, e.g. steel failure of fastener, concrete cone failure and pull-out failure of fastener, and welds failure. The concrete blowout failure is prevented because the anchor rods are embedded far enough from the concrete edges. The reinforcement of the concrete was designed to withstand the bending moment in the pad but no stirrups were added for transferring tensile forces tearing the concrete cone. The concrete pad was sufficiently wide and reinforced to resist splitting failure.

If the failure modes connected to concrete are not considered, the bending resistance around $y$ axis of the joint loaded with normal compressive force $F_v = 400$ kN is $M_{y,Rd} = 80$ kNm. The decisive component is the failure mode $F_{T1-2,Rd} = 189$ kN of T-stub in tension, containing two anchor rods. CBFEM shows 5% strain of the base plate at $M_y = 124$ kNm and occurrence of prying forces. The steel resistance of the fastener was determined as $N_{Rd,s} = 131$ kN, concrete cone resistance of a group of two anchors was $N_{Rd,c} = 221$ kN, and pull-out failure was $N_{Rd,p} = 245$ kN. Uncracked concrete was considered. The resistance of concrete cone failure mode is much higher compared to assessment according to ETAG, Annex C (72 kN). The decisive failure mode is the concrete cone failure, 110 kN per one anchor. This force in anchor was achieved when bending moment was
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\[ M_y = 119 \text{ kNm} \ (63 \% \text{ of the average of experimental bending resistance)} \] in case of uniaxial loading and \[ M = 83 \text{ kNm} \ (49 \% \text{ of the average of experimental bending resistance)} \] in case of biaxial loading.

**Figure 9:** Forces in anchor rods \( F_t \). CBFEM results are in solid lines, experiments are in dashed and dash-dotted lines.

### 4 Conclusion

EN 1993-1-8:2020 offers analytical models of anchor plate and fasteners in column bases which assure good and safe prediction for most basic structural solutions in a new face stressing easy to use of well approved models by engineering practice.

In new generation of structural Eurocodes after 2020 is expected to stress the principles of safety of applications of FEA by System response quantity process. The FEA analyses of the structural steel connections is replacing the curve fitting and component design methods. For its proper use is necessary to apply a good Validation and Verification procedures with well-defined hierarchy to allow a safe use. The presented results show the good accuracy of CBFEM verified to CM and to advanced calculations/experiments in cases where the CBFEM gives higher stiffness, resistance, or deformation capacity, see Ref.12.

The comparison with the experiment shows that the design method, based on determining the tensile forces in anchors using finite element method, gives safe results and accurately describes the behaviour of the joint.

### 5 Acknowledgement

The work was prepared under the R&D project MERLION II supported by Technology Agency of the Czech Republic, project No TH02020301 and by project No. LO1408 “AdMaS UP - Advanced Materials, Structures and Technologies”, supported by the Ministry of Education, Youth and Sports under “National Sustainability Programme I”.

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DESIGN OF MOMENT RESISTING REINFORCED CONCRETE CONNECTIONS USING POST-INSTALLED REINFORCING BARS

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ABSTRACT

The use of post-installed reinforcing bars for the realization of moment resisting connections between reinforced concrete beams and columns/walls or reinforced concrete columns/walls and foundations has increased significantly worldwide. The design of such connections with post-installed reinforcing bars is clearly limited by the use of straight reinforcing bars. Consequently, a clear understanding of the unique load-displacement properties of the post-installed reinforcing bars, the necessary loading and installation conditions, and the stresses introduced in the concrete members is required. In recent years, different design methodologies have been developed in Europe and in the United States to allow the design of such connections. In this paper, state of the art design methods based on Eurocode 2 and ACI 318 are compared and applied to a few case studies where post-installed reinforcing bar systems are used for moment resisting connections. Through the description of real-world examples, the challenges and the opportunities associated with the use of post-installed reinforcing bars and their proper design are explored.

1 Introduction

Post-installed reinforcing bars are generally used in concrete-to-concrete connections. The reinforcing bars are embedded in adhesive in a hole drilled into an existing concrete member and are cast in new concrete on the other side. Reinforcing bars installed in existing concrete are necessarily straight, while the portion embedded in new concrete can be straight or hooked.

Post-installed reinforcing bars are typically used to construct and extend existing reinforced concrete structural elements or to strengthen and rehabilitate them as shown in few examples in Figure 1.

While post-installed reinforcing bars are currently not addressed by the main reinforced concrete building codes in Europe (e.g. EN 1992-1-1⁶) and in the U.S.A. (e.g. ACI 318⁷), product qualification procedures⁸,⁹ that revolve around the concept of equivalence of performance between post-installed and cast-in reinforcing bars are available.

The European qualification procedures, as outlined in EOTA TR 023⁸ (to be replaced by the EAD 330087⁶), cover applications under static loading and fire exposure (EAD 330087⁶). Qualification procedures in the U.S.A. (AC308⁷), on the other hand, cover static and seismic loading applications.
A post-installed reinforcing bar system qualified according to EOTA TR 023\textsuperscript{e.5} or AC308\textsuperscript{e.7} can be typically used for the realization of lap splices, shear dowels, and anchorage of tensioned straight starting bars. In this latter case, the use of post-installed reinforcing bars is allowed by ACI 318\textsuperscript{e.4} as long as its development and splicing provisions for straight reinforcing are followed (e.g., the embedded reinforcing bar length is sufficient to reach steel yielding). EN 1992-1-1\textsuperscript{e.1}, on the other end, does not allow “the transmission of reinforcement force to concrete ties” and, thus, requires the use of a hooked end, which is feasible with cast-in reinforcing bars only. This is obviously the point around which the design of moment resisting connections revolves.

2 Design of moment resisting connections with post-installed reinforcing bars

The realization of moment resisting connections with post-installed reinforcing bar systems is quite common in engineering practice. Typical applications include retrofitting and/or extension of existing reinforced concrete structures (e.g. new columns/walls installed on existing foundations, new beams on existing columns/walls). While the design of such connections is possible in the USA using the provisions of ACI 318 Chapter 25\textsuperscript{e.4} for straight reinforcing (see Figure 2a), the required use of a hooked end as per EN 1992-1-1\textsuperscript{e.1} (see Figure 2b) precludes the use of post-installed reinforcing bars in Europe. Furthermore, it is interestingly noted that for steel-to-concrete connections, EN 1993-1-8\textsuperscript{e.3} allows the use of straight anchorage bars, e.g. adhesive anchors, designed according to CEN/TS 1992-4-5\textsuperscript{e.2} (see Figure 2c).

Clearly, different options are available to complete the design of a moment resisting connection with post-installed reinforcing bars. This article will focus on the following:

- Use of reinforced concrete theory (EN 1992-1-1\textsuperscript{e.1}). In this case, tension stresses cannot be transferred directly to concrete and strut-and-tie models need to be developed to predict the flow of forces in the system. An example is the model developed by Kupfer et al\textsuperscript{e.9} and validated by Hamad et al\textsuperscript{e.8}. This approach is briefly explained in Section 2.1.

- Use of anchor theory (EN 1993-1-8\textsuperscript{e.3} in conjunction with 1992-4\textsuperscript{e.2}). In this case, a design concept has been recently developed by Herzog\textsuperscript{e.11} (static loading) and Mahrenholtz\textsuperscript{e.10} (cyclic loading). This approach is described in Section 2.2.
Use of a combination of both reinforced concrete and anchor theories. Design of a connection following the development length (Chapter 25) or anchorage provisions (Chapter 17) of ACI 318\textsuperscript{e.4} could, in some cases, lead to significantly different results. Therefore, a method that tries to merge the philosophies included in ACI 318\textsuperscript{e.4} was developed by Charney et al\textsuperscript{e.12}. This approach is described in Section 2.3.

![Diagram of reinforced concrete column to foundation connection designed in accordance with ACI 318\textsuperscript{e.4}; b) Reinforced concrete column to foundation connection designed in accordance with EN 1992-1-1\textsuperscript{e.1}; c) Connection of steel column to foundation (EN 1993-1-8\textsuperscript{e.3})](image)

Figure 2: a) Reinforced concrete column to foundation connection designed in accordance with ACI 318\textsuperscript{e.4}; b) Reinforced concrete column to foundation connection designed in accordance with EN 1992-1-1\textsuperscript{e.1}; c) Connection of steel column to foundation (EN 1993-1-8\textsuperscript{e.3})

### 2.1 Design with strut-and-tie model (Kupfer et al.\textsuperscript{e.9})

EN 1992-1-1\textsuperscript{e.1} provisions require the use of bent bars in the existing concrete of moment resisting nodes. However, due to installation feasibility, only straight bars can be post-installed. A strut-and-tie model to design moment resisting connections with straight bars was developed based on experimental tests and numerical simulations by Kupfer et al.\textsuperscript{e.9} and further experimentally validated by Hamad et al.\textsuperscript{e.8}. The approach is based on the calculation of the reinforcing bar anchorage length such that concrete cone breakout is prevented by the development of compressive struts. The required anchorage length is function of the anchorage length and the angle (\(\theta\)) between the inclined compression strut and the horizontal direction. \(\theta\) must be between 30 and 60 degrees. More details are available in Hamad et al.\textsuperscript{e.8}. The steps needed to complete the design of a moment resisting connection using the method proposed by Kupfer et al.\textsuperscript{e.9} are summarized below.

Based on EN 1992-1-1\textsuperscript{e.1}, the internal design stress must be smaller than the design strength. The overall design can be divided in five steps.

a. Calculation per equilibrium of the forces acting in the node: \(V_1, V_2, V_3, N_1, N_2, N_3, M_1, M_2, M_3\).

b. Calculation of the anchorage length, \(l_b\), in accordance with EN 1992-1-1\textsuperscript{e.1}, section 8.4.4 or the adhesive product approval documentation reporting the relevant bond strength. The latter can be increased up to the full bond strength of the adhesive when an adequate confinement of the concrete is provided, Randl et al.\textsuperscript{e.14}.
c. Verification of the compression forces $D_{sd}$ acting on the concrete inside the moment resisting node. The design compressive stress, calculated per equilibrium and inclined by $\theta$, must be smaller than the resistance of the strut calculated by EN 1992-1-1 e.1, Equation (6.61).

d. Verification of the splitting forces acting on the concrete across the discontinuity region (D-region in accordance with EN 1992-1-1 e.1, Section 6.5):

$$\sigma_{sp} = \left( M_1 + \frac{(V_2 + V_3) \cdot z_1}{2} \right) \left( 1 - \frac{z_0}{z} \right) \left( 1 - \frac{l_b}{2 \cdot z} \right) \frac{2.42}{b \cdot z^2} \cdot f_{ct}$$

(splitting stress) (1)

Where $M_1, V_2$ and $V_3 =$ external forces on the node; $z_1$ is the inner lever arm outside the node region; $z_0$ is the inner lever arm inside the region; $z$ is the inner lever arm of slab section outside the node region; $l_b =$ required anchorage length.

$$z_0 = t_b - \frac{l_b}{2} - c_s$$

(lever arm inside the node) (2)

Where $t_b =$ drilled hole depth; $c_s =$ concrete cover to the center of the upper reinforcement of the base element.

e. Calculation of the horizontal action in the node:

$$H_{zd} = \left( M_1 + \frac{(V_2 + V_3) \cdot z_1}{2} \right) \left( 1 - \frac{z_0}{z} \right) + V_1 \cdot \left( \frac{z_1}{z_0} - 1 \right)$$

(3)

2.2 Design based on CEN/TS 1992-4-5$^2$ and Herzog, 2015$^{11}$

The design procedure developed and validated by Herzog$^{11}$ applies to column-foundation connections with post-installed reinforcing bars where the column is subjected to static loading and
is located far away from the edges. Based on this design procedure, all failure modes according to CEN/TS 1992-4.5\textsuperscript{2} shall be taken into account (see Eqs. (4) to (7)). Mahrenholtz\textsuperscript{10} subsequently proposed modifications to the existing formulas to account for the effect of cyclic loading. Note that this latter contribution is not discussed in this paper.

\[ N_{Rd,s} = A_s \cdot f_{yd} \]  
(steel yielding) \hspace{1cm} (4)

Where: \( A_s \) = cross sectional area of tensioned reinforcing bars; \( f_{yd} \) = design steel yielding resistance

\[ N_{Rd,c} = k_1 \sqrt{f_{ck,\text{cube}}} \cdot h_{ef}^{1.5} \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{re,N} \cdot \psi_{M,N} \]  
(concrete cone failure) \hspace{1cm} (5)

Where: \( k_1 = 7.2 \) or 10.1 for cracked or uncracked concrete respectively, \( f_{ck,\text{cube}} \) = concrete compressive strength measured on cube with 150 mm side length; \( h_{ef} \) = effective embedment length equal to the anchorage length of the reinforcing bar; \( \psi_{A,N} \) = factor for geometric effect of axial spacing and edge distance; \( \psi_{s,N} \) = factor for the disturbance of the distribution of stresses in the concrete due to the proximity of an edge of the concrete member; \( \psi_{re,N} \) = factor for the effect of dense reinforcement; \( \psi_{ec,N} \) = for the load eccentricity; \( \psi_{M,N} \) = the effect of a compression force between fixture and concrete in cases of bending moments with or without axial force according to Herzog\textsuperscript{11} (see Eq. (6)).

\[ \psi_{M,N} = 2.5 - z_1 / h_{ef} \geq 1.0 \]  
(6)

Where \( z_1 \) = inner lever arm according to Figure 3.

\[ N_{Rd,Np} = \pi \cdot d_b \cdot h_{ef} \cdot r_{Rd} \cdot \psi_{A,Np} \cdot \psi_{g,Np} \cdot \psi_{s,Np} \cdot \psi_{re,Np} \cdot \psi_{ec,Np} \]  
(combined bond/ concrete cone) \hspace{1cm} (7)

Where: \( \psi_{A,Np} \) = factor for geometric effect of axial spacing and edge distance; \( \psi_{g,Np} \) = factor for group effect for closely spaced bonded fasteners; \( \psi_{s,Np} \) = factor for the disturbance of the distribution of stresses in the concrete due to the proximity of an edge of the concrete member; \( \psi_{re,Np} \) = factor for the effect of dense reinforcement; \( \psi_{ec,Np} \) = for the load eccentricity.

### 2.3 Design based on ACI 318\textsuperscript{4} (Charney et al., 2012\textsuperscript{12})

As mentioned above, Charney at al.\textsuperscript{12} provide a design approach that takes into account the provisions of both ACI 318 Chapter 17\textsuperscript{4} (anchorage provisions) and ACI 318 Chapter 25\textsuperscript{4} (development length provisions). A brief explanation of the method is provided below (more details are available in Charney et al.\textsuperscript{12}). Firstly, the authors recognize that the reinforcing bar development length as per ACI 318\textsuperscript{4} Chapter 25, Equation (25.4.2.3a), is mainly intended to preclude concrete splitting (see Eq. (8)).

\[ I_d = \left[ \frac{1}{1.1 \lambda d} \cdot \psi', \psi_\epsilon' \right] \cdot d_b \]  
(SI-units) \hspace{1cm} (8)
Where: $f_y$ = steel yielding strength; $f_{c'}$ = cylindric concrete compressive strength; $\lambda$ = factor for lightweight concrete; $d_b$ = bar diameter; $c_b$ = minimum edge distance; $K_n$ = confinement term; $\psi_t$, $\psi_c$, and $\psi_s$ = factors for top reinforcement, epoxy coating and bar size, respectively.

Secondly, it is acknowledged that the design of adhesive anchors according to ACI 318$^4$ Chapter 17 accounts for a variety of failure modes including: steel failure, concrete breakout and bond failure.

Based on these two fundamental observations, Charney et al. $^{12}$ provide a solution for the following:

Design case 1 - near edge situations assumed to be controlled by splitting ($c_b < 6d_b$), which should be addressed using ACI 318$^4$ Chapter 25 (Equation (8)). In such applications, the embedment length is usually expected to be larger than $20d_b$ and the uniform bond stress assumption valid for adhesive anchors designed according to Chapter 17 is not applicable. Note that Equation (9) is applicable to all post-installed reinforcing bar systems qualified according to AC308$^7$.

Design case 2 - far from the edge applications where splitting does not play a significant role ($c_b \geq 1.5h_{ef}$), which could be addressed using ACI 318$^4$ Chapter 17. In this case the embedment length of the tensioned reinforcing bars shall satisfy Equation (9):

$$N_c \geq 1.25 \cdot A_y \cdot f_y$$  \hspace{1cm} (9)

Where: $N_c$ is the minimum of the strengths associated to concrete breakout ($N_{c_b g}$) and bond failure ($N_{a_b}$) mode calculated according to ACI 318$^4$ Chapter 17.

Design case 3 - intermediate situations between case 1 and case 2 (where the proximity to concrete edges could affect the concrete breakout and bond strengths), which should be addressed using both ACI 318$^4$ Chapter 17 and Chapter 25. Ultimately, the embedment length is the minimum of the values resulting from Equations (8) and (9). This procedure is usually iterative, in that the inclusion of factors to account for the influence of concrete edges in Equation (9), which are functions of the embedment depth, complicates the determination of a closed-form solution. Alternatively, a strength vs. embedment hierarchy diagram can be adopted to solve this design graphically (Pal$^{13}$).

3 Design case studies

Table 1 shows three real applications where post-installed reinforcing bar systems are used to design moment resisting connections.
### Table 1: Case studies

<table>
<thead>
<tr>
<th>Application*</th>
<th>Description</th>
</tr>
</thead>
</table>
| 1) Beam-Column Joint | The original beam was seriously damaged and removed. A new reinforced concrete beam needs to be connected to the existing column using post-installed reinforcing bars.  
- Beam dimension: 300 mm x 300 mm  
- Column dimension: 450 mm x 450 mm  
- Bending moment: 18.6 kNm  
- Shear: 34.4 kNm  
- Post-installed reinforcement: 3+3 \( \phi 20 \) mm  
- Concrete class C20/25 |
| 2) Beam-Wall Connection | In the execution phase, most of the planned reinforcing bars were not cast. Post-installed bars are needed to connect a new beam to the existing wall.  
- Beam dimension: 2200 mm x 1000 mm  
- Wall thickness: 2000 mm  
- Bending moment: 2.120 kNm  
- Shear: 1.500 kN  
- Post-installed reinforcement: 5+5 \( \phi 32 \) mm  
- Concrete class: C30/37 N/mm² |
| 3) Wall-Foundation Connection | A concrete wall needs to be connected to the existing reinforced concrete slab using post-installed reinforcing bars.  
- Wall dimension: 2000 mm x 800 mm  
- Foundation thickness: 1000 mm  
- Bending moment: 2000 kNm  
- Post-installed reinforcement: 5+5 \( \phi 20 \) mm  
- Concrete class: C20/25 |

*All dimensions in mm; N = new element; E = existing element

### 3.1 Comparison of design results

The three approaches described in Sections 2.1, 2.2, and 2.3 were used to complete the design of the case studies illustrated above and the results are shown in Figure 4. For reasons of simplicity, only the detailed calculations for design case 1) are provided in this paper. The following design assumptions were adopted:

- When possible, a strut angle (\( \theta \)) of 30° was used for the design calculations based on the strut-and-tie model proposed by Kupfer et al.⁹ (see Section 2.1). Note that 30° is the
minimum strut angle allowed by the method. The design calculations were conducted assuming a tension bond strength higher than the values required by EN 1992-1-1 Section 8.4.4 when possible (a proper concrete cover was ensured). Uncracked concrete conditions were assumed.

- The adhesive product used for the connection was assumed to be qualified for post-installed anchor and post-installed reinforcing bar applications. A product characteristic bond strength in uncracked concrete, $\tau_{Rk} = 15 \text{ N/mm}^2$, as taken from a post-installed anchor approval, was assumed. In all cases, the design was conducted assuming static loading and a standard base material temperature of approximately 20°C.

- For the design method described in Section 2.1, the following partial safety factors were assumed: $\gamma_c = 1.5$ for concrete-related failures and $\gamma_s = 1.15$ for steel-related failures. The design loads were determined following the requirements of EN 1992-1-1.

- For the design method described in Section 2.2 (CEN/TS 1992-4-5), the following partial safety factors were assumed: $\gamma_{mk} = 1.5$ for concrete-related failures and $\gamma_s = 1.2(f_y/f_{uk}) \geq 1.4$ for steel-related failures. The design loads were determined following the requirements of CEN/TS 1992-4-5. Uncracked concrete conditions were assumed.

- For the design method described in Section 2.3 (Charney et al.), the following partial safety factors were assumed for the calculations in accordance with ACI 318 Chapter 17: $\phi_c = 0.65$ for concrete-related failures and $\phi_s = 0.75$ for steel-related failures. For the calculations in accordance with ACI 318 Chapter 25 (development of steel yielding of the reinforcing), a partial safety factor $\phi_h = 1.00$ was used.

**General**

The design of the post-installed reinforcing bar connections was carried out to accommodate the design tension force, $N_{Sd}$, derived from the acting bending moment as follows:

$$N_{Sd} = M_{Sd} / z_1 = 18.6 \text{kNm} / \left[0.9(280 \text{mm} - 50 \text{mm})\right] = 82.7 \text{kN}$$

Where $z_1$ is the inner lever arm as shown in Figure 3.

**Design using strut-and-tie model (Kupfer et al.)**

The design is conducted calculating the length $l_b$ (see Figure 3) required to preclude a pullout failure under the design tension force $N_{Sd}$.

$$l_b = \max \left(I_{b, \text{min}} : (\sigma_{Sd} \cdot d_0) / (4 \cdot f_{bd})\right) = \max \left\{ 260 \text{mm}, (101 \text{N/mm}^2 \cdot 20 \text{mm}) / (2.59 \text{N/mm}^2 \cdot 4) \right\} = 260 \text{mm}$$

Where:

$$l_b = \min \left\{ 0.3 \cdot l_{b, \text{req}, f_{bd}} \cdot 10d_b \cdot 100 \text{mm} \right\} = \min \left\{ 0.3 \cdot (450 \text{N/mm}^2 \cdot 20 \text{mm}) / (2.3 \text{N/mm}^2 \cdot 4), 10 \cdot 20 \text{mm} \cdot 100 \text{mm} \right\} = 260 \text{mm}$$

Assuming a strut angle $\theta = 30^\circ$, the anchorage length of the post-installed reinforcing bars can be calculated as follows:

$$l_{bd} = z_0 + l_b / 2 + c_{\theta} = z_1 \cdot \tan \theta + l_b / 2 + c_{\theta} = 194 \text{mm} \cdot \tan 30^\circ + 255 \text{mm} / 2 + 40 = 280 \text{mm}$$

With $l_{bd}$ being the installation length of post-installed reinforcing bar. The assumed concrete strut is verified against the acting force $D_{Sd}$ in accordance with EN 1992-1-1, Equation (6.61).
\[ D_{sd} = N_{sd} / \cos \theta \leq D_{max} = \sigma_{\text{rd, max}} \cdot l_b \cdot \cos \theta = 551 / \cos 30^\circ \leq 10 \cdot 255 \cdot \cos 30^\circ = 637 \leq 2210 \text{kN/m} \]

Where: \( \sigma_{\text{rd, max}} = \nu' k_2 \cdot f_{ck} / \gamma_c = 1 \cdot 0.75 \cdot 20 \text{N/mm}^2 / 1.5 = 10 \text{N/mm}^2 \)

The tension force in the concrete is verified as follows:

\[
\max \sigma_{sp} = \left( M_1 + \frac{(V_2 + V_3) \cdot z_1}{2} \right) \cdot \left( 1 - \frac{z_0}{z_2} \right) \cdot \left( 1 - \frac{\ell_b}{2 \cdot z_2} \right) \cdot \left( \frac{2.42}{b \cdot z_2^2} \right) \leq \sigma_{t, c}
\]

\[
= 18.6 \cdot 10^3 \cdot \left( 1 - \frac{112}{450} \right) \cdot \left( 1 - \frac{255}{2 \cdot 450} \right) \cdot \left( \frac{2.42}{300 \cdot 450} \right) = 0.40 \leq 1.0 \text{N/mm}^2
\]

Where: \( \sigma_{t, c} = \alpha_{\text{ct}} \cdot f_{ck,005} / \gamma_c = 1 \cdot 1.5 \text{N/mm}^2 / 1.5 = 1.0 \text{N/mm}^2 \)

As shown above, an embedment length \( l_{bd} = 280 \text{mm} \) is needed to satisfy the design requirements.

**Design in accordance with CEN/TS 1992-4-5:2009\(^2\) and Herzog, 2015\(^11\)**

The design of the connection using anchor theory is conducted by determining the maximum embedment depth that satisfies the design requirements in case of concrete breakout and combined concrete cone and pullout failure. As a first step, a check on the steel strength is performed:

\[
N_{\text{rd,s}} = (A \cdot f_{uk}) / \gamma_{Ms} = \left( 20 \text{mm}^2 \cdot \pi \cdot 540 \text{N/mm}^2 / 4 \right) / 1.4 = 364 \text{kN} > 84.8 \text{kN} / 3 = 25.7 \text{kN} \text{ (steel failure)}
\]

The design concrete cone failure load satisfies the design when an effective embedment depth \( h_{ef} = 205 \text{mm} \) is considered:

\[
N_{\text{rd,c}} = k_1 \cdot \sqrt{f_{ck,\text{cub}}} \cdot h_{1.5}^{1.5} \cdot A_{c,N} \cdot A_{0,N} \cdot \psi_{s,N} \cdot \psi_{\text{re,N}} \cdot \psi_{\text{ec,N}} \cdot \psi_{M,N} / \gamma_{Mc} =
\]

\[
= 10.1 \cdot \sqrt{25N / \text{mm}^2} \cdot 205\text{mm}^{1.5} \cdot \frac{276,750 \text{mm}^2}{378,225 \text{mm}^2} \cdot 0.82 \cdot 1.0 \cdot 1.0 \cdot 1.34 / 1.5 = 83.3 \text{kN} > 82.7 \text{kN}
\]

Where \( A_{c,N}, A_{0,N}, \psi_{s,N}, \psi_{\text{re,N}}, \psi_{\text{ec,N}} \) are calculated according to CEN/TS 1992-4-5\(^2\) and \( \psi_{M,N} = 2.5 - z_{gk} / h_{ef} = 2.5 - 0.9 \cdot (300 \text{mm} - 35 \text{mm}) / 205 \text{mm} = 1.34 \) according to Herzog\(^11\). In the case of combined concrete cone and pullout failure, the design is satisfied with \( h_{ef} = 90 \text{mm} \) (minimum admissible embedment depth according to the European Technical Assessment).

\[
N_{\text{rd,Np}} = \pi \cdot d_b \cdot h_{ef} \cdot \tau_{bk} \cdot A_{p,N} / A_{0,N} \cdot \psi_{g,Np} \cdot \psi_{s,Np} \cdot \psi_{\text{re,Np}} \cdot \psi_{\text{ec,Np}} / \gamma_{Mc} =
\]

\[
= \pi \cdot 20 \text{mm} \cdot 90 \text{mm} \cdot 15 \text{N/mm}^2 \cdot 121,500 / 72,900 \cdot 0.94 \cdot 1.0 \cdot 1.0 \cdot 1.0 / 1.5 = 89.0 \text{kN} > 82.7 \text{kN}
\]

Where \( A_{p,N}, A_{0,N}, \psi_{g,Np}, \psi_{s,Np}, \psi_{\text{re,Np}}, \psi_{\text{ec,Np}} \) are calculated according to CEN/TS 1992-4-5\(^2\)

In conclusion, \( h_{ef} = 205 \text{mm} \) is needed to satisfy the design conditions as per CEN/TS 1992-4-5\(^2\).

**Design using Charney et al.\(^12\) (based on ACI 318\(^4\) Chapter 25)**

The reinforcing bar development length is controlling the design only when splitting controls (e.g., close edge conditions) and is determined in accordance with ACI 318\(^4\) Chapter 25 as follows:
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\[ l_d = \frac{f_y}{1.1 \lambda \sqrt{f_c} \left( \frac{\psi_c \cdot \psi_{c,N} \cdot \psi_{c,N} \cdot \psi_{c,N} \cdot \psi_{c,N}}{c_b + K_p} \right)} \quad d_p = \left[ \frac{1}{1.1 \lambda \sqrt{20N/mm^2}} \right] \cdot 20 \text{ mm} = 732 \text{ mm} \]

**Design using Charney et al.** (based on ACI 318^4 Chapter 17)

The design procedure based on the requirements of ACI 318^4 Chapter 17 as proposed by Charney et al.\(^{12}\) consists of selecting an anchorage length such that the nominal yielding strength of the reinforcing bar is exceeded by 25%. This method is applicable only to connections located sufficiently far away from the edges of the concrete member. In this paper, for the sake of comparison with the development length provisions, this design procedure is applied to all proposed examples.

\[ N_c \geq 1.25 \cdot f_y \cdot h_y = 1.25 \cdot 3 \cdot (20 \text{ mm})^2 \cdot \pi \cdot 450 N/mm^2 / 4 = 530 kN \]

Where:

\[ h_y(N_c) = \max(h_y(N_{cbg}), h_y(N_{ag})) \]

With:

\[ N_{cbg} = \phi_c \cdot k_c \cdot \sqrt{f_c} \cdot h_y \cdot A_{nc} \cdot A_{nc} \cdot \psi_{ec,N} \cdot \psi_{ed,N} \cdot \psi_{c,N} \cdot \psi_{cp,N} = \]

\[ = 0.65 \cdot 7.2 \cdot \sqrt{25} \cdot 750 \cdot 5.557 \cdot 500 \text{ mm}^2 \cdot 1 \cdot 1 \cdot 1.4 \cdot 1.0 = 538 kN > 530 kN \]

Being \( A_{nc}, A_{nc}, \psi_{ec,N}, \psi_{ed,N}, \psi_{c,N} \) and \( \psi_{cp,N} \) calculated according to ACI 318^4 Section 17, and

\[ N_{ag} = \phi_c \cdot \phi_{cr} \cdot \phi_{cr} \cdot \sqrt{f_c} \cdot h_y \cdot A_{na} \cdot A_{na} \cdot \psi_{ec,N} \cdot \psi_{ed,N} \cdot \psi_{cp,N} = \]

\[ = 0.65 \cdot 5 \cdot 0.81 \cdot 1.0 = 536 kN > 530 kN \]

Where \( A_{na}, A_{na}, \psi_{ec,N}, \psi_{ed,N}, \psi_{cp,N} \) are calculated according to ACI 318^4 Section 17.

In conclusion, the required embedment length is equal to maximum of the length corresponding to concrete breakout failure, \( N_{cbg} (h_y = 750 \text{ mm}) \) and pullout failure \( N_{ag} (h_y = 1350 \text{ mm}) \).

### 4 Conclusions

**Comparison of design results for all case studies**

Figure 4 shows the anchorage lengths calculated with the design methods presented in this paper. The columns shown with a striped hatch represent cases where the designs lead to unsatisfactory solutions because:

- The calculated anchorage length exceeds the depth of the base material available for installation of the post-installed reinforcing; or
- The calculated anchorage length is larger than $20d_b$ when the design is conducted using anchor theory.

More details on the proposed design case studies are provided below.

**Case study 1:**
Only design as per Sections 2.1 and 2.2 is possible. The design method proposed by Charney et al.\textsuperscript{12} shows that anchorage lengths exceed the member depth. This is clearly controlled by the need to reach rebar yielding. It is also worth noting that the design according to ACI 318\textsuperscript{4} Chapter 17 leads to an anchorage length almost twice the development length calculated according to Chapter 25 (1350 mm / 732 mm $\approx$ 1.85). This is mainly due to the different safety concept underlying the two methods (Chapter 17 and Charney et al.\textsuperscript{12}: 25\% steel overstrength and $\phi_c = 0.65$; Chapter 25: no steel overstrength and $\phi_s = 1.00$).

**Case study 2:**
In this case, only the strut and tie model proposed by Kupfer et al.\textsuperscript{9} and the development length equation according to ACI 318\textsuperscript{4} Chapter 25 are satisfying the design requirements. The visible difference in anchorage lengths provided by these two methods is mainly due to the conservativeness of the assumptions in the former (e.g., only the bottom portion of the embedded bar, $l_b$, is considered to be activated and to resist the applied load, as shown in Figure 3).

**Case study 3:**
In this case, three methods offer a feasible solution to the design requirements. Interestingly, the approach proposed by Charney et al.\textsuperscript{12} and the development length equation of ACI 318\textsuperscript{4} Chapter 25 lead to very similar embedments (750 mm and 727 mm, respectively). This is due to the more limited influence of edge distances in the calculation of the concrete breakout load compared to previous examples.

![Figure 4: Comparison of results for design case studies 1), 2), and 3)](image)
In conclusion, this paper clearly shows that multiple options can be adopted to satisfy the design requirements of real-case reinforced concrete moment resisting connections. The high variability and inconsistency of results, however, highlights the strong need for a unified approach capable of merging reinforcing and anchorage to concrete theories.

References:

4. ACI 318-14: Building Code Requirements of Structural Concrete (ACI 318-14) and Commentary, American Concrete Institute.
TOWARD A UNIFIED DESIGN APPROACH FOR STEEL STRENGTH OF ANCHORS IN STAND-OFF BASE PLATE CONNECTIONS

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ABSTRACT

It is common practice to level anchored steel base plate connections to concrete using leveling nuts, leaving a portion of anchor bolts exposed between the base plate and the concrete surface. The exposed portion may be filled with grout (e.g., for building columns) or left ungrouted (e.g., for cantilevered sign structures and railings). Despite the prevalence of use in practice, limited guidance for how to address stand-off in steel strength design of anchors is provided by building code entities. Existing provisions in specific concrete, steel, transportation, and energy industry guidelines are incomplete and, where guidance is given, is disparately addressed, often with highly conservative built-in assumptions, especially in the definition of bending parameters. As a result, the effects of stand-off are either ignored entirely or accounted for in an overly conservative manner. A recent dissertation by McBride14 thoroughly investigated steel strength of anchor bolts in both ungrouted and grouted stand-off base plate connections and design, including a survey of current code provisions. This paper compares the approaches of AASHTO1, ACI2, AISC3,4, ASCE5,6, and fib7 to identify disparities, gaps in design, and relationships to theoretical limits, illustrating the need for a unified design approach to address the steel strength of anchors in stand-off base plate connections. Preliminary recommendations for descriptive formulas, assumptions for treatment of bending, and code concepts are provided.

1 Introduction

Steel base plates connecting to concrete via anchor bolts often stand off of concrete surfaces to be leveled by way of 1) nuts threaded to the anchor supporting the base plate or 2) shim stacks between the base plate and the concrete surface. Following base plate installation and leveling, a grout pad may or may not be placed between the base plate and concrete surface. Figure 1 provides schematics of these types of stand-off base plate installations. Common connections utilizing ungrouted stand-off base plates with leveling nuts include sign, signal, and railing connections, while shim stacks and/or grout pads are often employed to complete column base connections in buildings. For the purposes of this paper, only ungrouted connections supported by leveling nuts (hereinafter simplified to “ungrounded connections”) and grouted connections are addressed, but the structural considerations for grouted connections can be applied to shimmed connections.
Despite the ubiquity of stand-off base plate connections and the obvious impacts on load transfer particularly in ungrouted connections, the impacts of stand-off are incomplete or wholly ignored in codes and design guidelines. While the topic has been addressed in small experimental studies using steel-to-steel connections and analytically, experimental work studying full connection behavior in concrete is not existent in ungrouted stand-off connections. Grouted research is limited to a small portion of one study, where grouted connections were subjected to cyclic loading and a more robust study by Gresnigt et al.

McBride executed a comprehensive test program investigating the steel strength of anchors in stand-off base plates, in part, to evaluate current methodologies. In this work, a wide range of stand-off base plate connections to concrete studied the influence of a wide array of experiments with the following variables: stand-off distance, connection type (i.e., ungrouted with leveling nuts, grouted with leveling nuts, grouted with a fiber-reinforced polymer wrap retrofit), anchor layout (circular/annular and rectangular base plates containing between one and six bolts), and combinations of axial and normal forces acting on anchors. As a background to the upcoming design recommendations stemming from the results of McBride, this paper summarizes the current state of design provisions, highlighting gaps and variations in properly addressing the effects of stand-off. Where applicable, tabular and graphical comparisons between existing design provisions that address steel strength of anchors in stand-off base plate connections.

2 Definition of forces

2.1 Force transfer from the connection to the anchors

Figure 2 illustrates the transfer of external forces acting on a cantilever sign structure into resultant forces on anchor bolts in a connection, demonstrating that all global (external) forces ultimately resolve into combinations of normal (tensile or compressive) and shear forces acting on individual anchors in stand-off base plate connections.
Subsequent descriptions of the transfer of global structural forces into component anchor forces include the assumptions that (1) the anchors comprising the group are all of the same material properties and cross-sectional dimensions with equal stand-off distances and fully restrained end conditions (i.e., all have the same bending and axial stiffnesses); and (2) global normal force $N_g$ and shear force $V_g$ act at the centroid of the anchor group.

The upper segment of Figure 2(c) demonstrates how the global direct shear $V_g$ and torsional moment $T_g$ combine to produce shear forces on individual anchors. Under the assumptions listed above, $V_g$ is resisted equally by all anchors in a common direction, while moment $T_g$ is resisted tangentially about the shear center of the bolt group, resulting in a unique magnitude and direction of shear for every bolt in the connection.

The lower segment of Figure 2(c) demonstrates how connection normal forces $N_g$ and overturning moment $M_g$ combine to produce axial forces on individual anchors. In ungrouted connections supported by leveling nuts, $N_g$ is resisted equally across all anchors, while $M_g$ resolves into axial anchor force in proportion to the distance from the neutral axis of bending of the bolt group. Grout and/or shim stacks, where present, contribute to the resistance of compressive forces in proportion to their stiffnesses relative to the bolts.

### 2.2 Cross-sectional forces on anchors in stand-off base plate connections

Figure 3 shows the possible forces on the exposed portion of an anchor cross-section in stand-off base plate connections for both ungrouted and grouted connections. For installations where leveling nuts are used and no grout pad is placed, e.g., in sign/signal structures and railings, the “exposed” portion of the anchor bolts necessarily carry all forces transferred from the base plate to the concrete; in addition to the resolved forces discussed in Section 2.1, bending moments are produced due to shear forces acting over the exposed length of the anchor bolt.
Shim stacks and/or grout pads between the base plate and concrete surface, when present, aid in supporting compressive and, as demonstrated in Figure 3 and will be elaborated upon in future publications, lateral support via combinations of clamping and friction action.

Figure 3: Simplification of shear transfer over the exposed length of an anchor bolt in an ungrouted stand-off base plate connection into shear forces and bending moments

2.3 Theoretical plastic limits on threaded circular steel cross-sections

The ultimate normal capacity of a steel cross-section is well accepted as the net tensile area multiplied by the ultimate engineering stress. While Yura et al.\textsuperscript{15} found that the definition of net tensile area is slightly incorrect, the error is small and Eq. (1) is well accepted in the anchor industry.

\[ A_{nt} = \frac{\pi}{4} \cdot \left( d_b - 0.9743 \right)^2 \]  

(1)

The theoretical shear capacity of steel is defined by the von Mises yield criterion\textsuperscript{16}, from which the application of purely shear reduces the criterion to the expression given in Eq. (2). The ultimate theoretical shear stress is equal to approximately 0.577 times the ultimate tensile stress. The 0.577 constant is typically simplified to 0.6 in structural engineering applications, which is then applied to the net tensile area with widespread experimental validation.

\[ \sigma_o = \sqrt{\sigma_x^2 + \sigma_y^2 + 3\tau_o^2} = \sqrt{3\tau_o^2} \]  

(2)

\[ \rightarrow \tau_o = \frac{1}{\sqrt{3}} \cdot \sigma_o \approx 0.577\sigma_o \]  

(3)

\[ \rightarrow V_o \approx 0.577\sigma_o A_{nt} = 0.577N_o \]  

(4)

The plastic bending moment capacity is related to the elastic bending capacity by a shape factor, \( \kappa \). For a circular cross-section, \( \kappa \) is equal to 1.7. McBride (2014) investigated the proper area within a threaded rod to assume bending, finding that the net tensile area significantly underestimates the bending capacity of a threaded rod. For simplicity, however, use of the net tensile area is recommended for the calculation of the section moduli.

\[ M_o = \sigma_o \kappa S = \sigma_o Z \]  

(5)

\[ S = \frac{\pi d_{nt}^3}{32} \]  

(6)
A steel cross-section’s ability to resist normal force, shear force, and bending moment relies on the interaction of such forces and can be described in interaction pairs. McBride (2014) demonstrated that for a circular cross-section, the interactions defined by Eqs. (7) through (9) define the theoretical limits, which will serve as a basis for comparison with existing design provisions in subsequent sections.

\[
\left(\frac{N_{\text{max}}}{N_o}\right)^2 + \frac{M_{\text{max}}}{M_o} = 1.0
\] (7)

\[
\left(\frac{N_{\text{max}}}{N_o}\right)^2 + \left(\frac{V_{\text{max}}}{0.577N_o}\right)^2 = 1.0
\] (8)

\[
\left(\frac{M_{\text{max}}}{M_o}\right)^2 + \left(\frac{V_{\text{max}}}{0.577N_o}\right)^{2.6} = 1.0
\] (9)

where:
- \(A_{nt}\) = calculated net tensile area of threaded rods
- \(d_{nt}\) = diameter back-calculated from the net tensile area of the bolt
- \(d_b\) = directional shear stresses on the element
- \(n\) = two-dimensional von Mises stress
- \(\sigma_o\) = ultimate normal stress capacity of material
- \(\tau_o\) = ultimate shear stress capacity of material
- \(\kappa\) = shape factor for bending = 1.7 for circular cross-section
- \(S\) = elastic section modulus for bending
- \(Z\) = plastic section modulus for bending
- \(N_{\text{max}}, V_{\text{max}}, M_{\text{max}}\) = maximum possible normal force, shear force, and bending moment in the presence of other forces
- \(N_o, V_o, M_o\) = fully plastic normal, shear, and bending moment capacities absent other forces

3 Design of ungrouted stand-off base plates supported by leveling nuts

Of the design code entities considered, three (AASHTO, ASCE, and \(fib\)) provide explicit provisions for design of anchor bolt steel strength in base plates supported by leveling nuts. Section 3.1 describes the provisions provided by these entities, then Section 3.2 translates the language into comparable terms.

3.1 Descriptions of design provisions

3.1.1 AASHTO LTS-6

AASHTO\(^1\) states that bending must only be considered when the distance between the concrete surface and the bottom of the leveling nut exceeds one bolt diameter, a distance at which McBride\(^14\) demonstrates that approximately 40% of the pure shear capacity is lost. Bending is assumed to occur between the concrete surface and the bottom of the leveling nut.

Bending stresses are added to the normal stresses with the maximum allowable stress equaling the yield; in precise terms, this translates to an assumed linear interaction between normal forces and bending moments. Because there is a limit on the maximum stress, it is inferred that the cross-section
is not allowed to move beyond the elastic bending limit. The interaction between shear stresses and normal stresses takes the circular relationship defined in Eq. (8). Allowable shear stresses are taken as 60% of the allowable tensile stresses.

### 3.1.2 ASCE 48 and ASCE 113

ASCE 48⁵ and ASCE 113⁶ address steel design of anchor bolts by requiring enough area to independently take up the normal force, shear force, and bending moment on the anchor cross-section. Such a summation of areas is analogous to conservative linear interactions between normal force, shear force, and bending moment.

Bending is assumed to occur between the top of the concrete surface and the top of the leveling nut and washer; however, the point of contraflexure is assumed to occur at 5/8 of the bending distance to account for different bending stiffnesses between concrete and steel boundary conditions. As with AASHTO provisions, bending capacity is conservatively taken as the elastic limit. Also as with AASHTO, bending is not required to be considered with an exposed length less than one anchor diameter. Oddly, the nominal shear capacity is equivalent to the tensile capacity; the only factor that distinguishes the two capacities is a 5% difference in the load resistance factors.

### 3.1.3 fib Bulletin 58

fib⁷ presents the shear resistance of anchors in ungrouted stand-off base plates following recommendations within Scheer et al.⁸ It is readily seen that this equation describes the shear strength as the minimum of the bending moment capacity and the shear capacity of the anchor. The assumed plastic shape factor in the definition of bending capacity is 1.5.

The interaction of the normal force with the bending moment due to shear is taken as linear, but the interaction of direct shear (more specifically, the presence of shear stresses on the cross-section) with normal force and shear is not considered in the stand-off provisions. Nevertheless, with the assumed lever arm for bending moment, as will be shown in subsequent figures, the calculation of available shear force is very conservative.

### 3.2 Summary of findings by McBride

McBride¹⁴ found that it is appropriate to calculate the interaction of shear force, normal force, and bending moment in anchor bolt steel design with stand-off base plates. For convenience when expressing the interaction of all forces simultaneously, the interaction equations given in Section 3.3.2 were recommended. Theoretical limits described in this document matched the data well when bending moment was calculated over a distance equal to the exposed length plus one-half of a bolt diameter to account for concrete spalling. Bending moment resistance fit best with an assumed diameter between that corresponding to the net tensile diameter and the gross diameter; it should be noted that the calculation of bending moment resistance is proportional to the cube of the assumed diameter and must not be taken as the gross diameter. It was found that the proper combination of anchor ductility and ratio of tensile to shear force precluded the need to calculate the effects of bending moment, significantly reducing the demand on the anchor.
3.3 Comparisons between design provisions

3.3.1 Definition of bending terms

Figure 4 shows the assumed lever arms for the codes considered. Because varying exposed lengths disproportionately affect the moment calculations between design provisions, an assumed exposed length of one bolt diameter and base plate thickness of one bolt diameter are chosen to allow for one-to-one comparison. Figure 5 depicts the available capacity with pure shear loading based on the bending assumptions provided by various design provisions.

The calculated proportion of bending demand to capacity depends on (1) the definition of the effective lever arm \( l_{eff} \) over which bending occurs and (2) the assumed shape factor \( \kappa \) defining the relationship between elastic and plastic bending capacity. In Table 1, \( l_{eff} \) and \( \kappa \) are compared between codes, the theoretical limit, and the recommendations based on the findings of McBride. Finally, in Table 1, a normalization factor \( \rho_l \) is introduced, which is useful in comparing the effective bending moment capacities between design provisions in Table 2.

![Figure 4: Lever arm definition comparisons between codes](image1)

![Figure 5: Effect of exposed length on available capacity with pure shear loading](image2)

**Table 1: Normalization of bending terms via \( \rho_l \)**

<table>
<thead>
<tr>
<th></th>
<th>( l_{eff.i} )</th>
<th>( \kappa_i )</th>
<th>( \rho_l )</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO</td>
<td>( l_{ex} = d_b )</td>
<td>1.0</td>
<td>0.59</td>
</tr>
<tr>
<td>ASCE</td>
<td>( l_{ex} + d_b = 2d_b )</td>
<td>1.0</td>
<td>0.24*</td>
</tr>
<tr>
<td>fib</td>
<td>( 0.5d_b + l_{ex} + d_b + 0.5t_p = 3d_b )</td>
<td>1.5</td>
<td>0.29</td>
</tr>
<tr>
<td>theoretical limit</td>
<td>( l_{ex} = d_b )</td>
<td>1.7</td>
<td>1.0</td>
</tr>
<tr>
<td>recommended</td>
<td>( l_{ex} + 0.5d_b = 1.5d_b )</td>
<td>1.7</td>
<td>0.67</td>
</tr>
</tbody>
</table>

*This value also accounts for the unique assumption of the point of inflection of bending occurring at 5/8 of the lever arm.

where:
- \( l_{ex} \) = exposed length of the bolt; given as one bolt diameter for comparison
- \( l_{eff.i} \) = effective lever arm defined by individual code
- \( \kappa_i \) = inferred shape factor for determining ultimate bending moment capacity
- \( \rho_l = (l_{ex}/l_{eff})(\kappa_i/1.7) \) = normalization factor to compare ultimate moment capacities against the theoretical limit with \( l_{ex} = d_b \) assumed
3.3.2 Interaction between design forces

Table 2 and Figure 6 show how AASHTO, ASCE, and fib provisions for double-nut connections compare against each other and the theoretical limit in the three bivariate relationships between shear force, bending moment, and normal force. Because different design paradigms are employed, these comparisons are stripped of safety factors (e.g., load amplification factors and capacity reduction factors due to uncertainty) to reveal the assumptions about nominal capacity underpinning the respective design provisions. Limits on the available shear forces in Figure 6 are based on the maximum possible shear force given a one-diameter exposed length. To provide a uniform basis for comparison, the following definitions are employed in defining the interaction equations:

\[ n = \frac{N_{\max}}{N_o} \] = proportion of maximum permissible shear to normal capacity given the presence of bending moment or normal force
\[ v = \frac{V_{\max}}{N_o} \] = proportion of maximum permissible shear to normal capacity given the presence of bending moment or normal force
\[ m = \left( \frac{V_{\max} \cdot l_{ex}/2}{\rho \cdot M_0} \right) \] = proportion of maximum permissible bending moment caused by applied shear to the shear with assumed exposed length of one diameter.

Table 2: Inferred interaction equations between shear forces, bending moments, and normal forces for ungrouted stand-off base plates with assumed exposed length of one bolt diameter

<table>
<thead>
<tr>
<th>interaction between</th>
<th>shear force and bending</th>
<th>shear force and normal force</th>
<th>bending force and normal force</th>
</tr>
</thead>
<tbody>
<tr>
<td>AASHTO</td>
<td>( \left( \frac{V_{\max}}{0.6N_o} \right)^2 + \left( \frac{V_{\max}/2}{0.59M_o} \right)^2 = 1.0 )</td>
<td>( \left( \frac{V_{\max}}{0.6N_o} \right)^2 + \left( \frac{N_{\max}}{N_o} \right)^2 = 1.0 )</td>
<td>( \frac{V_{\max}/2}{0.59M_o} + \frac{N_{\max}}{N_o} = 1.0 )</td>
</tr>
<tr>
<td>ACI</td>
<td>no specific design provisions for ungrouted stand-off connections</td>
<td></td>
<td></td>
</tr>
<tr>
<td>AISC</td>
<td>no specific design provisions for ungrouted stand-off connections</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ASCE</td>
<td>( \frac{V_{\max}}{0.85N_o} + \frac{V_{\max}/2}{0.24M_o} = 1.0 )</td>
<td>( \frac{V_{\max}}{0.85N_o} + \frac{N_{\max}}{N_o} = 1.0 )</td>
<td>( \frac{V_{\max}/2}{0.24M_o} + \frac{N_{\max}}{N_o} = 1.0 )</td>
</tr>
<tr>
<td>fib</td>
<td>no specific design provisions for interaction with shear on a cross-sectional level for stand-off connections</td>
<td></td>
<td>( \frac{V_{\max}/2}{0.35M_o} + \frac{N_{\max}}{N_o} = 1.0 )</td>
</tr>
<tr>
<td>recommended</td>
<td>( \left( \frac{V_{\max}}{0.6N_o} \right)^2 + \left( \frac{V_{\max}/2}{0.67M_o} \right)^2 = 1.0 )</td>
<td>( \left( \frac{V_{\max}}{0.6N_o} \right)^2 + \left( \frac{N_{\max}}{N_o} \right)^2 = 1.0 )</td>
<td>( \frac{V_{\max}/2}{0.67M_o} + \left( \frac{N_{\max}}{N_o} \right)^2 = 1.0 )</td>
</tr>
<tr>
<td>theoretical limit</td>
<td>( \frac{V_{\max}}{0.6N_o} ) ( 2.6 + \left( \frac{V_{\max}/2}{M_o} \right)^2 = 1.0 )</td>
<td>( \left( \frac{V_{\max}}{0.6N_o} \right)^2 + \left( \frac{N_{\max}}{N_o} \right)^2 = 1.0 )</td>
<td>( \frac{V_{\max}/2}{M_o} + \left( \frac{N_{\max}}{N_o} \right)^2 = 1.0 )</td>
</tr>
</tbody>
</table>

Figure 6: Bivariate interaction diagrams between shear force, bending moment, and normal force corresponding to Table 2
4 Design of grouted stand-off base plates

4.1 Descriptions of design provisions

4.1.1 AASHTO LTS-6
AASHTO\textsuperscript{1} states that grout installed beneath the base plate may not be designed to contribute to connection strength in double-nut connections (those using leveling nuts). Thus, two double-nut connections, one with and one without a grout pad, would be designed using the beam model described above.

4.1.2 ACI 318
ACI\textsuperscript{2} states that 80\% of the shear strength of an anchor group is maintained in the presence of a grout pad irrespective of grout pad height (base plate stand-off distance). It can be inferred that this provision applies to both pure shear and combined tension and shear applications, where an elliptical interaction equation with tension and shear exponents of 5/3 is assumed.

4.1.3 AISC Design Guide 1
Beyond stating in commentary that anchor bolt bending must be considered when shear is transferred through base plate bearing against anchor bolts, AISC specifications\textsuperscript{3} do not provide guidance for ungrouted or grouted stand-off base plates. However, AISC Design Guide 1\textsuperscript{4} provides an example using the interaction of tension forces in the anchor from direct tension and bolt bending with shear forces in the anchor in a base plate supported by a grout pad. Their calculation uses the same beam bending model as in AASHTO\textsuperscript{1}, but takes the length of the beam as the distance from the top of the grout to the middle of the grout pad. This assumes that anchor bolt length within the thickness of the grout pad does not contribute to bolt bending and also implies that grout pads shall not contribute to anchor bolt strength.

4.2 Summary of findings by McBride
McBride\textsuperscript{14} developed extensive theoretical relationships between applied forces, stand-off distance, and assumed friction coefficients that matched a wide range of tests varying these parameters. Interestingly, the interpretation of ACI\textsuperscript{2} provisions above provided accurate predictions (predicting approximately 25\% higher strength than test results with a 16\% COV against test results) despite its relative simplicity.

4.3 Comparisons between design provisions
The three codes that address the strength of grouted connections, AASHTO\textsuperscript{1}, ACI\textsuperscript{2}, and AISC\textsuperscript{3,4}, do so in such disparate ways that even a qualitative comparison is difficult. A future publication will compare the test data from McBride\textsuperscript{14} against these provisions.
5 Conclusions

The following conclusions are drawn from this study of existing provisions for steel capacity of anchors in stand-off base plate connections:

1. Codes that do not address stand-off distance leave a widespread application entirely to the engineering community in both ungrouted and grouted stand-off connections.
2. It is inappropriate to neglect stand-off distance of any magnitude in ungrouted connections supported by leveling nuts. Such allowances should be omitted from existing provisions.
3. Interactions between shear force, normal force, and bending moment in ungrouted stand-off connections are treated highly variably between existing codes, especially in the effective bending moment capacity based on an assumed lever arm and plastic capacity shape factor, resulting in moderately to highly conservative treatment of anchor bending. Standardized provisions would allow for a uniform factor of safety across disciplines.
4. Existing provisions for grouted stand-off base plate connections are limited and vary not only in magnitude, but in concept altogether, with disagreement about the presence/location of bending and the ability of the grout pad to contribute to strength.

6 Recommendations for code applications

Future publications will summarize data from McBride\textsuperscript{14}. Nevertheless the following recommendations are drawn from the findings in McBride (2014) for ungrouted stand-off base plates:

1. Consider the interaction of shear force, normal force, and bending moment in anchor bolt steel design with stand-off base plates.
2. Apply the shape for a circular section of 1.7 to a section modulus calculated with the diameter corresponding to the net tensile diameter. The minimum root diameter may be conservatively used, but the gross bolt diameter must not be used.
3. To determine bending moment demand, impose the shear demand over an effective length equal to the exposed length plus 0.5 anchor diameters to account for concrete spalling effects.
4. A combination of anchor ductility and ratio of tensile to shear force may preclude the need to calculate the effects of bending moment, significantly reducing the demand on the anchor.
5. Interaction equations for steel strength should not be mixed in a “final” interaction equation with other failure modes for ungrouted

Further analysis is needed to determine the best design approach for grouted stand-off base plates. If an immediate solution is desired, however, it is recommended that the existing ACI 318 provisions be refined to explicitly account for the interaction between shear and tensile forces, then incorporating additional language to utilize the positive effects of friction due to compressive forces on the connection. Furthermore, the importance of fully grouting with non-shrink, highly flowable grout should be emphasized in provisions allowing this practice.
7 Recommendations for future work

Opportunities for future work include finite element investigations verifying and expanding upon current results, investigation of seismic and fatigue performance, investigations of the influence of base plate flexibility, and retrofit solutions for deficient stand-off base plate connections.

8 Acknowledgements

The authors gratefully acknowledge financial support by the National Science Foundation and the Florida Department of Transportation. Physical research assistance was provided by Andrew De Alba, Kunal Malpani, Joshua Burkard, and the late Hubert “Nard” Martin, whose years of service to the University of Florida and impact on the success, development, and morale of so many students will carry forward immeasurably. You are and will be missed, Nard.

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5. ASCE 48: Design of steel transmission pole structures, American Society of Civil Engineers (ASCE), 2011.


EN 1992-4 – THE LONG ROUTE TO A EUROPEAN STANDARD FOR FASTENING TO CONCRETE

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ABSTRACT

This paper outlines the general approach of the new Eurocode 1992-4 for design of fastening to concrete in the EN 1992 series for structural concrete, its accompanying CEN Technical Reports and bureaucratic side effects which had major impact on the publication of this standard. It covers cast-in-situ headed fasteners and mechanical and chemical post-installed as well as anchors anchor channels subjected to static, quasi-static, fatigue and seismic actions. The proposed design procedures are in general harmony with provisions of ETAG 001 and ACI 318.

While EN 1992 has had rules for the design of reinforced concrete structures since 1994, it has been silent on the subject of fastening to concrete until 2001 when the working group CEN/TC 250/SC 2/WG 2 was formed and 2009 when the CEN/TS 1992-4 series on the design of fastenings for use in concrete were first published. These rules represented improved provisions of the ETAG 001 design approach. Then ETAG 001 was updated so that two nearly identical design concepts could be used for the design of fasteners. In 2011 CEN performed a systematic review of the CEN/TS 1992-4 series. As a result Committee CEN/TC 250/SC 2 decided that a harmonized design procedure for fastening to concrete is required which should be published in 2014 as a single EN 1992-4 document. The design concept requires product prequalification according to the corresponding EOTA guideline.

Now, 16 years after the initial kick-off the challenge of developing a new Eurocode for the design of fastenings is virtually finished, at least in its actual phase. This paper describes where Eurocode 1992-4 is and its general approach. While the new provisions cover much and represent a great improvement, there are still steps on the way of further developments.

1 Introduction and background

The international harmonization of the design of fasteners for use in concrete has a long history. In 1987 the CEB TG III/5 ‘Fastenings to Reinforced Concrete and Masonry’ was formed. With the contributions of experts from all over the world the knowledge in anchorage to concrete could be collected and the cornerstone for an international database with results of tests with fasteners was laid. In 1991 the CEB Bulletins 206 and 207 ‘Fastenings to reinforced concrete and masonry structures’, State-of-art reports, Parts 1 and 21 were published. These state of the art reports served
as background for the start of the international discussion on the design of fastening to concrete in Europe. In the early 1990s the Concrete Capacity (CCD)-Method for the design of fastenings was developed. It was published in 1995\(^2\). In Europe it is termed Concrete Capacity (CC)-Method. It is based on the so-called extended Kappa-Method\(^3,4\). The CC-Method is innovative, because it visualizes the \(\kappa\)-factors in the extended Kappa method and is very user friendly. It takes into account all load directions and failure modes. In 1993 the CC-Method was nationally implemented in Germany in a DIBt-design guideline\(^5\) and in 1997 it was accepted by CEB (CEB Bulletin 226)\(^6\).

However, the process of the development of a European design procedure for fasteners was not only driven by the academic community but also from the politicians in Europe. By implementing the European Construction Products Directive in 1989\(^7\) a single European market for construction products was to be created, bringing down trade barriers for products and simplifying rules for engineers to enable everyone in the European Union to have direct access to 28 countries and almost 500 million people.

The EN Eurocodes and EOTA ETAGs were expected to contribute to the establishment and functioning of this common market and were meant to lead to more uniform levels of safety in construction in Europe serving as a framework for drawing up harmonized technical specifications for construction products (ENs and ETAs). These provisions were intended as means to prove compliance of building and civil engineering works with the basic requirements of the Construction Products Directive, particularly Basic Requirement 1 ‘Mechanical resistance and stability’ and Basic Requirement 2 ‘Safety in case of fire’.

CEN/TC 250 ‘Structural Eurocodes’ was formed to standardize structural and geotechnical design rules for building and civil engineering works taking into account the relationship between design rules and the assumptions to be made for materials, execution and control. However, in 1991, when the academic community was in the stage of creating standardized design rules for anchorage to concrete subcommittee CEN/TC 250/SC2 ‘Design of Concrete Structures’ was already overloaded with Eurocode 2 ‘Design of Concrete Structures’. To not jeopardize the schedule of the first publication of Eurocode 2 their primary focus was on the European harmonization in the design of the structures. Fastening to concrete was on a lower priority level and therefore not intended to be part of the first version of Eurocode 2. This was when EOTA, the European Organization for Technical Approvals, decided to jump in.

In addition to its primary task to develop the provision ETAG 001 ‘Guideline for European Technical Approval of Metal Anchors for Use in Concrete’\(^8\) EOTA WG 6.01 ‘Anchors’ started to develop design provisions based on CEB Bulletin 226 ‘Design of Fastenings to Concrete’\(^6\) covering the design of headed cast-in and post-installed anchors. Finally, in 1997 EOTA published ETAG 001, Annex C ‘Design Method for Anchorages’\(^9\) which is valid only for fasteners covered by a European Technical Approval (ETA) with the intent to convert this Annex to a CEN standard in the short term.
2 Development of EN 1992-4

2.1 General
Eurocode 2 (EN1992-1-1)\textsuperscript{10} announced in Section 2.7 a standard on the design of fastenings for use in concrete where the performance of fasteners should comply with a CEN standard or should be demonstrated by a European Technical Approval. Therefore it was necessary to fulfill this requirement and to replace ETAG, Annex C as soon as possible. As first step, it was intended to only make the most important improvements to ETAG, Annex C and to use this text as the first CEN design standard for fastenings to obtain internationally harmonized and accepted design procedures for most types of fasteners.

Before the work on a European standard can be started some bureaucratic hurdles have to be passed. First CEN/TC 250/SC 2 had to agree to start a Working Group ‘Design of Fastenings’ (WG) to prepare a standard. After the first approach to CEN/TC 250/SC 2 in 1998 asking for implementation of basic requirements for the design of fastenings in Eurocode 2 subcommittee CEN/TC 250/SC 2 decided in 2000 with Resolution 123 to develop a standard ‘Design of Fastenings for Cast-in-situ and Post-installed Systems’. The initial scope was ‘design of fastenings with cast-in-situ and post-installed systems applied to transfer concentrated loads into concrete structures and to connect prefabricated components’. After formal approval by the main committee CEN/TC 250 which is in charge of ‘Structural Eurocodes’ CEN-members were requested to nominate experts by July 31, 2000. Prof. Eligehausen was appointed by CEN/TC 250/SC 2 as convenor of this working group. To assist this standard development process DIN volunteered to provide the secretariat. However, travel and drafting costs had to be carried. After having found sponsors from the fastening technology industry and so passing also this hurdle the work could be started.

2.2 CEN/TS-series
The first meeting of CEN/TC 250/SC 2/WG 2 ‘Design of Fastenings’ was held in Stuttgart, Germany still in late 2000. The experts came from ten nations and represented four different parties (Figure 1). After fixing the content and structure of the future standard as well as the work program a time table was developed indicating that a draft of the standard should be dispatched to the CEN Management Center (CMC) for CEN-Enquiry in April 2003 and that the new EN on the design of fastenings should be available in December 2004 i.e. only about four years after the first meeting. All experts agreed on the concept, that the field of application of the design procedures should be limited to fasteners prequalified based on a European Technical Specification such as a harmonized European Product Standard (hEN) or a European Technical Approval Guideline indicating the technical data required for the corresponding design models. This approach is summarized in Figure 2.
From the first meetings it was obvious that the question on how to consider the condition of the structural member serving as base material – cracked or uncracked – should be taken into account in design was one of the most significant challenges to be resolved by WG2. As usual in these cases a Task Group was formed in spring 2002 to develop a CEN/Technical Report with the working title ‘Effect of Cracking’ to give information on

- cracking in concrete (background information),
- effect of cracking on the load-bearing behavior of fasteners,
- consideration of cracks by European Technical Approvals (ETAs),
- definition of non-cracked concrete, and
- simplified rules for the definition of non-cracked concrete.

After extreme efforts over nearly three years documented in six drafts of the Technical Report the task group stopped its work. The TG members saw no way to come up with reliable simplified rules to ensure uncracked concrete serving as anchorage material of the corresponding structural component over the service life of a fastening. Therefore this Technical Report was never published.
Until January 2003, in the first seven meetings of WG2 five drafts were developed indicating topics which could not be resolved shortly. Time consuming examples were:

- EN 13670 ‘Execution of Concrete Structures’\(^1\) does not cover the execution of fasteners. However, resistance and reliability of fastenings are significantly influenced by the manner in which they are installed. Therefore a chapter on installation of fasteners had to be included. The discussions on the content and extent of this chapter took its time.
- Removal of the design of sockets and shear lugs since not enough supporting data was available.
- Significant improvements of the design methods for adhesive anchors, anchor channels and resistance to fire.
- Design of fasteners under seismic loading is not covered by EN 1998\(^{12}\). Co-ordination with CEN/TC 250/SC 8 was necessary.
- Design of fastenings supported by grout pads should also be covered by EN 1993\(^{13}\) but based on different research in a different way. Both approaches had to be brought in line in a coordinated process.

Therefore, at the meeting of CEN/TC 250/SC 2/WG 2 in January 2003 it was decided to update the target dates for the release of the design standard based on the assumption that the delay would be about 1.5 years. Two options were presented:

1) To publish the new standard as planned as EN 1992-4 the publication would be delayed until December 2006. This was also related to the meeting schedules of the superior committees CEN/TC 250 and SC2 which have to agree to the work of WG2, or

2) to publish the design procedure for fastenings as CEN/Technical Specification (CEN/TS) which was formerly named European Pre-Standard (prEN). Since for a CEN/TS only the acceptance of SC2 is required the CEN/TS could be released at the end of 2004 presumed that the final draft would be finalized by WG2 by December 2003 and sent to SC2 directly afterwards. Disadvantage of this procedure is, that a European Memberstate can but does not have to implement a CEN/TS. On the other hand the designers in practice could use a ‘Prestandard’ concluding satisfactory design procedures for most applications in a short time. The acceptance is ensured by a link of EOTA to the ETAs. During the lifetime of a Technical Specification which is usually about two years the design provisions could be improved based on the user’s experience and proposals. Furthermore open questions in the application and design of fasteners could be resolved by WG2 during this period. These improvements could be included in the final draft of the EN until 07/2006 for a release of the EN in 01/2008.

WG2 decided to proceed with option 2) and CEN/TC 250/SC2 accepted to the proposal in 2003 and approved the publishing of the final draft of "Design of Fastenings for use in concrete" as a Technical Specification by the middle of 2004. CEN/TC 250/SC 2 wished to resolve open questions and to produce a harmonized EN version of the pre-standard to be carried out during life of the Technical Specification. SC 2 opted for a three year lifetime instead of the usual two years to be sure to receive a clear impression of the acceptance of the new design rules in practice.
In the meantime Draft 7 was finalized, but not all design rules were included and it consisted already close to 100 pages. Since standards are in many members states very expensive it was decided to split the design procedures into one part ‘General’ which gives basic provisions and four product specific parts. So a designer needs only the Part ‘General’ and the product specific Part for the design (Figure 3). Part 1 ‘General’ consisted of 58 pages, Part 2 ‘Headed Fasteners’ of 33 pages, Part 3 ‘Anchor Channels’ of 26 pages, Part 4 ‘Post-installed Fasteners – Mechanical Systems’ of 27 pages and Part 5 ‘Post-installed Fasteners – Chemical Systems’ of 26 pages.

Figure 3: Interaction between the different parts of the CEN/TS

In 2006, parallel to the development of design procedures by CEN/TC 250/SC 2/WG 2 the draft Technical Report ‘Design and use of inserts for lifting and handling of precast units’ was produced by CEN/TC 229/WG 4, however, with a different design approach for the same type of anchorage (Figure 4). Since WG2 was of the opinion that concrete does not care if a headed fastener is used as permanent fastener or only for a couple of lifting operations and therefore has the same resistance WG2 and SC2 were of the opinion that the same design procedures should be used independent of the field of application and agreement between the two sets of documents should be achieved.

Figure 4: Lifting insert with head for anchorage into concrete

CEN/TC 229, having been informed on the CEN/TC 250/SC 2 approach was very surprised. They pointed out, that lifting devices covered by the CEN/TC 229 draft CEN/TR are placed during casting in reinforced (non-cracked) concrete with a defined amount of extra reinforcement as specified in the
CEN/TR. There are other differences also such as the type and size of insert, and the fact that lifting inserts are generally not permanently loaded. In addition, fixings are usually inserted under site conditions with little control, while lifting inserts are placed under factory conditions with quality controls. The two applications are thus very dissimilar and differences in the method of design are not only to be expected, but are indeed necessary. Therefore the first version of CEN/TR 15728\textsuperscript{14} released in 2008 was different from the CEN/TS 1992-4 design models. After time consuming discussions in the TC229-TC250-Adhoc group, installed to harmonize differences occurring in both committees, it was decided to remove lifting inserts from the CEN/TS scope. Then there is no conflict between both documents any more. Besides, it should be mentioned that the revised version of CEN/TR 15728\textsuperscript{14}, released in March 2016 was corrected and refers to the CEN/TS provisions.

Finally in December 2007 the Final Drafts of the five Parts of the CEN/TS 1992-4\textsuperscript{15} were sent to Formal Vote. After a short revision of the documents necessary due to the comments received after the two months Formal Vote CEN/TS Parts 1 to 5 were published in August 2009, i.e. more than four years after the intended date. Due to lack of manpower in the CEN Management Center (CMC) the last stage between initialization of the Formal Vote and publication took nearly 20 instead of six months.

The different stages of the development are summarized in Figure 5.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{fig5.png}
\caption{Development of CEN/TS 1992-4}
\end{figure}

It was not December 2004 and not an EN 1992-4, but nearly five years later the European Pre-standard CEN/TS 1992 ‘Design of Fastenings for Use in Concrete’ was published at least. That meant that the work had to continue.

The new CEN/TS 1992-4 provisions were widely accepted. Due to their benefits they quickly became mandatory for the design of headed studs and anchor channels whereas for the other products EOTA provisions still remained applicable.
2.3 EN 1992-4

Since the tendency to bureaucracy in EOTA is not as distinct as with CEN, EOTA could continuously improve and publish its design guidelines based on the technical content of the CEN/TS drafts so that the designers always could work with the up to date design approach agreeing with future CEN provisions (Figure 6) since the actual European Approvals always referred to these documents.

![Figure 6: Development of CEN/TS 1992-4 and EOTA design provisions](image)

In the meantime it became obvious that the political environment for construction products in Europe would also change. The European Parliament replaced on March, 9 2011 the Construction Products Directive (CPD) 89/106/EEC by the Construction Products Regulation (CPR),(EU) No 305/2011 laying down harmonized conditions for the marketing of construction products. From this resulted that EOTA’s function switched from the European Organization for Technical Approvals (EOTA) to the European Organization for Technical Assessment (still EOTA) with the effect that the work of EOTA was limited to assessment procedures now. The design procedures for the safe use of the construction products should come from other sources such as CEN. A design standard was needed with the enforcement of the European Technical Assessments (ETAs) for construction products in 07/2013.

Therefore CEN/TC 250/SC 2 decided in May 2011 to reactivate the CEN/TC 250/SC/WG 2 dealing with ‘Fasteners’ to develop proposals for the revision of the CEN/TS 1992-4-series and the conversion into ENs. Objective of the revision should be to eliminate everything of ‘textbook’ character from the CEN/TS parts and to significantly reduce the volume.

In December 2011, following the CEN rules CEN/TC 250/SC 2 had initiated the ‘Systematic Review’ procedure on the CEN/TS 1992- 4 series i.e. the European countries were asked on their experience with the CEN/TS 1992-4 series.
The result of the systematic review indicated that

- three countries considered the CEN/TS 1992-4 series as best possible solution,
- nine countries were of the opinion that improvement is necessary,
- eleven countries preferred a revision of the documents, and
- two countries wanted that cast-in parts should be regulated by CEN/TC 229 provisions and post-installed anchors be regulated by EOTA. However, the latter proposal is not in agreement with the CPR regulations.

Based on this decision CEN/TC 250/SC 2/WG 2 recommended the following actions for the revision of the CEN/TS 1992-4-Series:

- The five parts of CEN/TS 1992-4 should be shortened and merged into one volume,
- the volume should be limited to about 80 pages,
- the structure should be based on the verification of action/resistance deviating from CEN/TS concept with product orientation
- sections of minor importance to the designer in everyday work should be deleted,
- the Annexes A ‘Load Transmission’, D ‘Fire’, and E ‘Seismic’ should be written in a way so that they could be included in EN 1992-1-1, EN 1992-1-2 and EN 1998, respectively in a later stage.

In February 2012 CEN/TC 250/SC 2 authorized WG2 with the continuation of their work based on their proposal.

WG2 immediately started its work and proposed the first best case time frame for the development of the new standard as given in Figure 7. With this time frame it would have been possible to nearly meet EOTA’s initial schedule which indicated first ETAs based on the CPR referring to EN 1992-4.

![Figure 7: First time frame for the development of EN 1992-4](image-url)
The first draft of the new EN 1992-4 document from February 2012 had 80 pages and was difficult to read. In addition the work for the development of the CEN/TRs on ‘Seismic design’, ‘Anchor channel additions’, ‘Design of Plastic anchors’, ‘Design according to the Theory of Plasticity’, ‘Design of Fastenings with Anchor Reinforcement’, ‘Design of Fastenings for Redundant (non-structural) Systems’ and ‘Installation Safety Aspects in design of Fastenings’ started in Task Groups. The intent was to publish the CEN/TRs giving additional and explanatory information together with EN 1992-4.

Though the reinstated WG2 consisted of 38 members from 15 CEN Member states the work had to be focused. In March 2013, six months and five meetings behind the initial schedule the seventh and final draft of EN 1992-4, now consisting of 98 pages and serving as base for the CEN Enquiry was finalized. Then, it was decided to prioritize the CEN/TRs covering ‘Redundant non-structural Systems’, ‘Anchor channels – supplementary Rules’ and ‘Plastic design with headed and post-installed Fasteners’. The work with respect to the other CEN/TRs was put on hold. At that time WG 2 thought that a publication by mid of 2014 would be realistic.

Finally, after translation of the final draft in the languages French and German and reformatting by the CEN secretariat, which caused the volume to increase to 117 pages, the CEN Enquiry started in September 2013 – about one year later than assumed in the best case scenario. The end of the Enquiry in February 2014 resulted in 922 comments from nine countries. 46% of these comments were of technical, 39% of editorial and the rest of other nature. This meant the publication in 2014 was not feasible.

The amount of comments could not be handled within one WG2 meeting with all participants. Therefore the TG ‘Preparation Panel’ was founded to respond to each single comment in detail. It was clear that the TG needed more time to first internally discuss all comments and to modify the draft accordingly. As a result, CEN/TC 250 agreed to a nine month tolerance after CEN Enquiry to avoid the cancellation of the project and to get more time for the ‘Preparation Panel’ to observe the comments. In December 2014, eleven meetings and numerous web conferences later the final draft of EN 1992-4 could be presented to CEN/TC250/SC 2. The corresponding CEN/TRs were also finalized. Now the last steps to the publication of EN 1992-4 and its accompanying CEN/TRs could be tackled.

On March 4, 2015 CEN/TC 250/SC 2 gave the approval to ask CEN/TC 250 to initiate the Formal Vote procedure for EN 1992-4 and the corresponding procedures for the CEN/TRs as developed by CEN/TC 250/SC 2/WG 2. CEN/TC 250 agreed on May 12, 2015 and by the end of May 2015 WG2 was informed by the CEN secretariat that the Formal Vote procedure could be started. CEN/TC 250 informed on August 24, 2015, that all CEN/TRs were approved by the National Members and could be published at the same time as EN 1992-4. Then, beginning of September 2015 the CEN secretariat announced that EN 1992-4 was editorially improved. The reason was that the CEN Technical Board had decided in 2014 to change from the ‘Arial’ font to ‘Cambria’ for European standards. The document provided by WG2 was written in ‘Arial’. The WG2 ‘Preparation Panel’ should have a quick look at the reformatted document with respect to typographical errors so that the Formal Vote could start on September 27, 2015 …and now the drama took its course:
After a short pilot check many errors were detected by the ‘Preparation Panel’. Therefore they had to decide that a full and proper review of the ‘editorially improved’ document was necessary. The complete review resulted in the detection of several hundred faults mainly, formatting e.g. inconsistent font type and size for symbols, Notes, inconsistent forms of tables… etc. An example is given in Figure 8. The WG2 ‘Preparation Panel’ finalized its work after several WebEx-Meetings on October 19, 2015. Afterwards the ‘editorially improved’ EN 1992-4 version with readable handwritten WG2 comments was sent out for further improvement to CEN.

The CEN-secretariat revised EN 1992-4 and a CEN editor performed ‘editorial’ changes so that the document complied with CEN rules. This document was sent to Formal Vote without further review by CEN/TC 250/SC 2/WG 2. Finally, the CEN BOSS information system announced the Formal Vote in May 2016. However, the start of the Formal Vote was in September 2016 with a two months commenting phase, so that the Formal Vote process was finalized beginning of November 2016 i.e. 18 months after the approval by CEN/TC 250.

The Formal Vote ended with 23 approvals – seven of them with reservations -, eight abstains and one negative which could be interpreted as a ‘yes with reservation’. This meant that EN 1992-4 was approved. The Formal Vote resulted further in 67 pages of editorial comments from eight member states. These comments had to be reviewed within CCMC (CEN CENELEC Management Center) and the editorial comments incorporated into the text. Then EN 1992-4 could have been published. However, the EN 1992-4 version ‘editorially improved by the CEN secretariat and editors did not technically agree with the EN 1992-4 approved for Formal Vote by CEN/TC 250 in May 2015 which caused ‘yes with reservation’ and ‘no’ votes during the Formal Vote process:

The ‘editorial’ improvements made by CEN staff resulted partially in technical changes. These changes yielded possible misinterpretations and a change of the scope of EN 1992-4. Examples after ‘editorial’ improvement written in italic are:

- Post-installed ribbed reinforcing bars used to connect concrete members are covered by a European Technical Product Specification. This EN applies when connections are designed in accordance with EN 1992-1-1.

  However, EN 1992-4 does not cover post-installed rebars, and connections cannot be designed according to EN1992-1-1!

- EN 1992-4 only considers fasteners for fastening statically indeterminate non-structural systems...

  This text limits the use of EN 1992-4 to redundant fasteners only which is nonsense since also statically determinate structural fastenings are covered by EN 1992-4

- …
FprEN 1992-4:2015 (E)

Figure 8: Editorially improved version of EN 1992-4 and marked-up errors - example

<table>
<thead>
<tr>
<th>Concrete related failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete cone failure, concrete edge failure, concrete blow-out failure and concrete pry-out failure</td>
</tr>
<tr>
<td>$Y_{MC,0} = Y_{M,0} \cdot Y_{R,0}$ for seismic repair and strengthening of existing structures see EN 1998</td>
</tr>
<tr>
<td>$Y_{R,0} = 1,5$ for headed fasteners and anchor channels satisfying the requirements of 4.6 (in tension and shear)</td>
</tr>
<tr>
<td>$\geq 1,0$ for post-installed fasteners in tension, see relevant European Technical Product Specification</td>
</tr>
<tr>
<td>$= 1,0$ for post-installed fasteners in shear</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Concrete splitting failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Y_{MC} = Y_{M,0}$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Pull-out and combined pull out and concrete failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Y_{MC} = Y_{M,0}$</td>
</tr>
</tbody>
</table>

4.4.2.2 Ultimate limit state (static and seismic loading)

1. Partial factors for fastenings under static and seismic loading shall be applied to characteristic resistances.

2. The recommended values for the partial factors for fastenings under seismic loading are identical to the corresponding values for static loading. For accidental loads the partial factors according to Table 4.1 are recommended.

NOTE The value of a partial factor for use in a Country under static, seismic and accidental loading may be found in its National Annex, when the partial factor is not product dependent. The recommended values are given in Table 4.1. They take into account that the characteristic resistance is based on $f_{ck}$, except for $f_{ck}$ should be used for bending of the channel of anchor channels and steel failure of supplementary reinforcement.

4.4.2.3 Ultimate limit state (fatigue loading)

Partial factors for fastenings under fatigue loading $Y_{MC,0} \cdot Y_{M,0} \cdot Y_{R,0}$ and $Y_{MC,0} \cdot Y_{M,0} \cdot Y_{R,0}$ shall be applied to characteristic resistances.

NOTE The values of the partial factors for fastenings under fatigue loading for use in a Country may be found in its National Annex. It is recommended to take the partial factor for material as $Y_{MC,0} = 1,35$ (steel failure), $Y_{M,0} = Y_{M,0}$ (concrete related failure modes).

4.4.2.4 Serviceability limit state

The partial factor for resistance is $Y_{M,0}$ and shall be applied to characteristic resistances.

NOTE The value of the partial factor for serviceability limit state for use in a Country may be found in its National Annex. For the partial factor $Y_{M,0}$ the value $Y_{M,0} = 1,0$ is recommended.
For the members of the CEN technical committees it was evident that the EN 1992-4 version passing the Formal Vote could not be published. Corrections were necessary to

- resolve misinterpretations,
- remove unclear and confusing wording,
- avoid contradictions within the document,
- clearly define the range of application of the design method.

Therefore in November 2016 CEN/TC 250/SC 2 decided to contact CCMC with the intent to delete the technical modifications of the CEN-Editor and to put the technical content of EN 1992-4 back to the version of May 2015. However, this petition was dismissed. Therefore it was proposed to ask the CEN Technical Board for permission of the necessary changes prior to publication. This intention was rejected by the CEN executive since there was an approved version of EN 1992-4 ready for publication. They interpreted the CEN rules such that technical changes to a ‘published’ standard could only be approved by a Formal Vote procedure. Therefore, now, in September 2017 there is still no EN 1992-4 available and the start of the second Formal Vote has to be awaited. Now the publication of EN 1992-4 is expected for mid of 2018 i.e. 20 years after the first steps in this direction were undertaken.

However, there was and is still the need for publicly available documents for the design of fasteners in concrete. To bridge this time span EOTA was active and published the design provisions ETAG 001, Annex C\textsuperscript{9}, EOTA TR 029\textsuperscript{17}, EOTA TR 045\textsuperscript{18} and EOTA TR 047\textsuperscript{19} with design methods complying with the EN 1992-4 version approved for Formal Vote in May 2015. These documents will be withdrawn when EN 1992-4\textsuperscript{20} is published.

3 Conclusions

The idea to create a standard on design of fastenings was developed in 1998, a working group installed in 2000 and a first series of pre-standards, the CEN/TS 1992-4 series finally published in 2009. Now, eight years later, the simple transition from the CEN/TS 1992-4 series to the EN 1992-4 is not finalized yet. However, the authors are confident, that the publication will be realized in 2018.

The authors learnt that with a better focus on the CEN rules for drafting CEN standards, the focus on the limited content absolutely necessary for a standard (e.g. no textbook additions), the unambiguous clarity of the language developed by the experts and last but not least a better communication between WG2 and CCMC (CEN CENELEC Management Center) staff, e.g. the CEN editors as well as CEN management the processing time for the development of EN 1992-4 could have been significantly shortened.

Last year CEN has improved its standard development process. Then a standard should be developed in 27 months after assignment of the Work Item. The authors wish that this time frame will be kept for all future standards – the development of EN 1992-4 will take nearly ten times this time frame...
4 Acknowledgement

The authors would like to thank all contributors to the committee work in WG2 and SC2 for their never ending support.

References:


ON THE LIMITATIONS OF THE CURRENT DESIGN PROVISIONS OF PREN 1992-4 FOR DESIGN OF ANCHOR GROUPS SUBJECTED TO TENSION LOADS

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ABSTRACT

The current design method for anchorages given in prEN1992-4\textsuperscript{1} is primarily based on the Concrete capacity design (CC-design) method\textsuperscript{2}. The design of anchor groups according to the provisions of prEN 1992-4 is limited to certain regular configurations and is rather conservative in many cases. The design of only rectangular anchor groups with regularly spaced anchors with maximum three anchors in a row is allowed. The steel plate which connects the anchors must be sufficiently rigid according to the provisions, provided that the deformations of the base plate compared to the vertical displacement of the single anchors are negligible and the linear strain distribution will be valid. However, no explicit criterion defines the rigidity. Furthermore, certain influences such as the influence of load eccentricity, moment loading etc. are considered in a way that they are not applicable to groups if more than two anchors in a row are applied. Additionally, the design rules for the evaluation of the failure loads of anchor groups in cracked concrete are generally over-simplified by assuming all anchors located in cracks. However, this can be on the unsafe side in certain cases.

In this study, pull-out tests in cracked and non-cracked concrete were carried out on anchor groups with diverse configurations covered by prEN 1992-4. The tests focused on the influence of eccentric loading, loading on the concrete edge, anchor plate thickness and cracked concrete in particular in the case of concrete cone failure, to emphasize the above discussed limitations of the code. The results from the experiments were compared to the calculated ultimate loads according to prEN 1992-4. The comparison showed that the current code gives either conservative or non-conservative approximation for the ultimate capacity of the anchor groups for the investigated cases. Therefore, there is a need to develop such a design concept that is applicable for the design of anchor groups, which accounts for the above influences.

1 Introduction

The prEN1992-4 is part of the series Eurocode 2: Design of concrete structures, which provides a design method of post-installed and cast-in fasteners for use in concrete. The prEN 1992-4 uses the fastener design theory i.e. to transfer the loads into the concrete component, the tensile capacity of the concrete is directly used. The design of fastenings according to prEN 1992-4 is based on the Concrete capacity design (CC-design) method\textsuperscript{2} and takes different load directions and failure modes.
into account. The load directions tension and shear loading as well as their interaction and all relevant failure modes (steel failure, concrete cone failure, pull-out failure, splitting under tension loading and steel failure, concrete edge failure, pry-out under shear loading) are required for the verification in the ultimate limit state (ULS). In the serviceability limit state (SLS), the anchor displacements under relevant loading actions should not be larger than the admissible displacement. According to prEN 1992-4 only rectangular anchor groups with regularly spaced anchors with maximum three anchors in a row are allowed, where the group consists of fasteners of same type and size i.e. it is assumed that all anchors in a group exhibit approximately the same stiffness. The possible configurations are categorized according to fastener groups with or without hole clearance, edge distances and loading directions. Figure 1 shows the permissible configurations of fastenings (post installed and cast in headed fasteners) according to prEN 1992-4. The design of the fixture or steel plate which connects the anchors within a group is not covered. Requirements for the steel plate to be stiff are stated but explicit criterion for the rigidity is not defined. However, the verification of the rigid base plate is very important because in the case of non-rigid base plate the deformation of the plate has an influence on the lever-arm of the internal forces. The calculated internal lever-arm in this case is greater than in reality and therefore, the calculated ultimate group capacity is on the unsafe side. Furthermore, in the case of eccentric loading and/or if bending moment is acting on the group, the design according to the CC-design method is assumed to be not sufficient for groups with more than 2 anchors in a row because the influences are considered in a way that not more than two anchors in a row are applied. The condition of the concrete (cracked or non-cracked) must be determined by the design engineer. When the crack initiation in the concrete member cannot be ruled out in the anchorage zone, the design of single anchors as well as anchor groups (including all anchors of the group) must be carried out assuming all anchors in cracked concrete according to prEN 1992-4.

![Figure 1: Permissible configurations according to prEN 1992-4: a) fastenings without hole clearance for all edge distances and fastenings with hole clearance situated far from edges (c_i ≥ max \{10h_{ef}, 60d_{nom}\}) for all load directions and fastenings with hole clearance situated near to an edge (c_i < max\{10h_{ef}, 60d_{nom}\}) loaded in tension only; b) fastenings with hole clearance situated near to an edge (c_i < max\{10h_{ef}, 60d_{nom}\}) for all load directions](image)

2 Scope

The aim of this study was to investigate the concrete cone capacity of diverse anchor group configurations under tension with particular interest on the influence of eccentric loading, concrete edge, anchor plate stiffness and the influence of the cracked concrete. This case was also investigated when only some anchors of the group were located in cracks. The results of the pull-out tests were compared to the calculations according to prEN 1992-4 to emphasize the differences between the test results and the code. The corresponding test programs are summarized in Table 1 - Table 3.
3 Experimental investigations

3.1 Testing concept and test program

In this study, quasi-static pull-out tests were performed on anchor groups with different anchor configurations according to Table 1 - Table 3 to investigate the concrete cone capacity. The experimentally investigated configurations are covered within the scope of current design provisions according to prEN 1992-4. For the experiments, a torque-controlled expansion anchor with reduced embedment depth and two different injection systems with relatively high bond strength (Epoxy mortar + M12 or M16, 12.9 threaded rod) were used. The test parameters such as concrete strength, anchor diameter, embedment depth and anchor spacing (group test) were chosen in the way that other failure modes than concrete cone failure i.e. steel failure, pull-out failure or bond/mixed failure in the case of bonded anchors did not occur. In this paper, the eccentricities, e1 and e2 are defined as distances between the point of the load application and the center of gravity of all anchors within the group in two perpendicular directions 1 and 2, respectively.

Table 1: Test program – non-cracked concrete – expansion anchor (Loading point marked by x)

<table>
<thead>
<tr>
<th>Anchor system</th>
<th>Test identification</th>
<th>Test type</th>
<th>Mean concrete cube strength $f_{cm}$ [N/mm²]</th>
<th>Embedment depth $h_{ef}$ [mm]</th>
<th>Anchor spacing $s_1$ [mm]</th>
<th>Anchor spacing $s_2$ [mm]</th>
<th>Edge distance $c$ [mm]</th>
<th>Eccentricity of loading $e_1$ [mm]</th>
<th>Eccentricity of loading $e_2$ [mm]</th>
<th>Base plate dimensions $s_x<em>y</em>t$ [mm]</th>
<th>Number of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torque-controlled expansion anchor – M12</td>
<td>G01R</td>
<td>-</td>
<td>25.8</td>
<td>55</td>
<td>98</td>
<td>-</td>
<td>&gt;c_{cr}</td>
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<td></td>
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<td></td>
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<tr>
<td></td>
<td>G11R</td>
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<td>25.8</td>
<td>55</td>
<td>98</td>
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<td>&gt;c_{cr}</td>
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<td>0</td>
<td>270<em>30</em>30</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>G12</td>
<td>-</td>
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<td>Bonded a. – M16</td>
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</table>

Table 2: Test program – non-cracked concrete – bonded anchor (type 1)

<table>
<thead>
<tr>
<th>Anchor system</th>
<th>Test identification</th>
<th>Test type</th>
<th>Mean concrete cube strength $f_{cm}$ [N/mm²]</th>
<th>Embedment depth $h_{ef}$ [mm]</th>
<th>Anchor spacing $s_1$ [mm]</th>
<th>Edge distance $c$ [mm]</th>
<th>Eccentricity of loading $e_1$ [mm]</th>
<th>Eccentricity of loading $e_2$ [mm]</th>
<th>Base plate dimensions $s_x<em>y</em>t$ [mm]</th>
<th>Number of tests</th>
</tr>
</thead>
<tbody>
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<td>Bonded a. – M16</td>
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<td>70</td>
<td>120</td>
<td>&gt;c_{cr}</td>
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<td>0</td>
<td>400<em>120</em>50</td>
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</table>
### Table 3: Test program – cracked concrete – bonded anchor (type 2)

| Anchor system | Test identification | Test type | Cracked (\(\Delta w = 0.3\)) / non-cracked | Mean concrete cube strength \(f_{cm} [N/mm^2]\) | Embedment depth \(h_e [mm]\) | Anchor spacing \(s_1 [mm]\) | Edge distance \(e [mm]\) | Eccentricity of loading \(e_1 [mm]\) | Eccentricity of loading \(e_2 [mm]\) | Base plate dimension \(x \times y \times t [mm]\) | Number of tests |
|----------------|---------------------|-----------|------------------------------------------|---------------------------------|-----------------|----------------|----------------|----------------|----------------|----------------|----------------|----------------|
| Bonded anchor – M12 | G61 | n-n-n | 23.0 | 60 | 120 | 0 | 0 | 400*120*50 | 4 |
| | G62R | c-c-c | 0 | 0 | 3 |
| | G62 | n-c-n | 0 | 0 | 3 |
| | G63 | c-c-n | 0 | 0 | 3 |
| | G64 | c-n-n | 0 | 0 | 3 |
| | G67 | c-n-c | 0 | 0 | 3 |
| | G65 | c-n-n | -60 | 0 | 3 |
| | G66 | c-n-n | -60 | 0 | 3 |
| | G68 | c-n-c | -60 | 0 | 3 |

### 3.2 Test setup

The anchors were installed according to the corresponding Manufacturer’s Installation Instructions. To investigate the discussed influences properly, an accurate positioning of the anchors within the group was necessary. Steel templates with pilot-holes were built with accurate hole spacing to ease the precise vertical drilling process and to keep the bonded anchors upright during the hardening. The tests were carried out in concrete members with a compressive strength range of 23 N/mm² – 66.6 N/mm² (Tables 1 – 3). The crack width in the case of the respective tests in cracked concrete was uniformly \(\Delta w = 0.3\) mm. The concrete mixture of the corresponding concrete batches was designed according to DIN EN 206⁴. The compressive strength was measured on concrete cubes (\(a=150\) mm) according to DIN EN 12390-1⁵. The group tests were carried out using a tests setup, which consisted of a tension test rig with adequate hydraulic testing cylinder, load cell, displacement transducers and a data interface. The clear support distance of the test rig was greater than \(4h_e\) measured from the side anchor of the anchor group to allow the formation of an unrestricted concrete cone. The applied loads, the vertical anchor displacement, the anchor plate displacement at the point of load application and the crack widths were measured and recorded continuously. To transfer the load into the anchor plate, internally threaded holes were drilled into the anchor plates (see Figure 2). In the case of tests with eccentrically applied loads, a special hinge provided the free rotation of the anchor plate (rotation-unrestrained anchor plate). Thus, the load which was acting on the group could be distributed among the individual anchors based on their stiffness (in the case of same type and size of anchors, the stiffness was influenced by the cracked concrete condition).

![Figure 2: Test setup for diverse configurations in non-cracked and cracked concrete](image-url)
3.3 Test results

According to the test program (Table 1 - Table 3) pull-out tests were carried out on anchor groups far from concrete edge and near to the edge using 1x3, 1x2 and 2x2 anchor configurations. To emphasize the limitations of the code, the ultimate concrete cone capacity of the tested configurations was calculated according to prEN-1992-4. The test results are given as relative values in Figure 4 - Figure 6 and in Table 4. All executed tests failed by a common concrete cone and the typical failure modes are shown in Figure 3.

![Figure 3: Failure modes of diverse configurations in cracked and non-cracked concrete](image)

4 Comparison of test results with prEN 1992-4

4.1 General

The test results were compared to the calculated concrete cone capacities according to prEN 1992-4. In the following graphs (Figure 4 - Figure 6), the mean ultimate loads are shown as values relative to the case in which the anchor group is positioned far from edge and centrically loaded with an eccentricity of e/s. In Section 4.2 - 4.6 the diverse influences i.e. influence of eccentricity, influence of concrete edge and influence of cracked concrete on the concrete cone capacity of anchor groups are discussed in detail.

![Figure 4: Influence of (1x) eccentric loading and anchor plate stiffness](image)

![Figure 5: Influence of concrete edge and (1x) eccentric loading](image)
4.2 Influence of eccentricity

The influence of the eccentric loading on the ultimate concrete cone capacity of anchor groups is well known. However, the influence of loading eccentricity is currently considered in the design provisions in a way that it is not applicable to groups with more than two anchors in a row. Therefore, to investigate the influence of the eccentricity in the case of anchor groups far from concrete edge with more than 2 anchors in a row tests were carried out (groups with 1x3 configuration). The results of the 1x3 eccentric tests on bonded anchors (G52, G53 and G54) and on torque-controlled expansion anchors (G13) demonstrated that the calculation of the ultimate concrete cone capacity according to prEN 1992-4 comes up with conservative results. It can be seen in Figure 4 and Figure 5 that a larger eccentricity (of internal forces) of the loading leads to a higher difference between the calculation and test results. According to prEN 1992-4, the influence of the eccentric loading is taken into account by the factor $\psi_{ec,N}$ (Equation 1), which considers when different tension loads are acting on the individual anchors within the group and is calculated as follows:

$$\psi_{ec,N} = \frac{1}{1 + 2 \frac{e_N}{s_{cr,N}}}$$  \hspace{1cm} (1)$$

where $e_N$ is the eccentricity of the resulting tension force of tensioned fasteners to the center of gravity of the tensioned fasteners; $s_{cr,N}$: characteristic center to center spacing of fasteners to ensure the characteristic resistance of the individual fasteners under tension load. If no other influences i.e.
reinforcement or concrete edge etc. is present, the characteristic concrete cone resistance of an eccentrically loaded anchor group is calculated according to Equation 2.

\[ N_{Rk,c} = N_{Rk,c}^0 \cdot \frac{A_{c,N}}{A_{c,N}^0} \cdot \psi_{ec,N} \]  

(2)

where \( N_{Rk,c}^0 \): characteristic concrete cone resistance of a single fastener not influenced by adjacent fasteners or concrete edges; \( A_{c,N}^0 \): reference projected area; \( A_{c,N} \): actual projected area, limited by overlapping concrete cones of adjacent fasteners (\( s<s_{ct,N} \)) and by concrete member edges (if \( c<c_{ct,N} \)).

There are applications for tension loaded anchor groups, where the eccentricity of loading is about two axes. In this case, according to prEN 1992-4, the factor \( \psi_{ec,N} \) is to be determined separately for each direction (\( \psi_{ec,N,1} \), \( \psi_{ec,N,2} \)) and both factors shall be inserted in Equation 2 (\( \psi_{ec,N} = \psi_{ec,N,1} \cdot \psi_{ec,N,2} \)). It is assumed that the multiplication of the factors \( \psi_{ec,N} \) leads to conservative results. To confirm this, the influence of the biaxial eccentricity was investigated (see test series G31R, G32). Figure 6 shows that due to the multiplication of the factors (\( \psi_{ec,N,1} \), \( \psi_{ec,N,2} \)), the results according to prEN 1992-4 are conservative. Due to biaxial eccentricity, the ultimate concrete cone capacity of the group compared to centric tests (G31R) was reduced by only 27% in the tests and by 41% according to the CC-design method. Therefore, there is a need to perform further tests of this kind up to 3x3 anchor configurations with different e/s relations to investigate the influence in detail.

Figure 7: Anchor group with load eccentricity in two directions

4.3 Influence of anchor plate stiffness
The influence of anchor plate stiffness on the concrete cone capacity was investigated on eccentrically loaded anchor groups (see test series G53, G54). The 1x3 configuration was tested with two different anchor plate stiffness with \( t=50 \text{ mm} \) (\( t/s =0.4 \)) and with \( t=25 \text{ mm} \) (\( t/s=0.2 \)). Figure 4 shows that in the tests, due to decrease in the anchor plate thickness, the reduction of the ultimate capacity was 12%, but these results are still above the calculated values according to prEN 1992-4, where a rigid plate was assumed.

4.4 Influence of concrete edge
Pull-out tests on anchor groups with 1x2 (G01R, G02) and 1x3 (G11R, G12) anchor configurations were carried out. If anchor groups are positioned near to the concrete edge and are loaded in tension, the stress disturbance near to edge is taken into account by the factor \( \psi_{s,N} \). Furthermore, the geometrical influence of the overlapping adjacent failure cones (if \( s<s_{ct,N} \)) as well as the presence of
the concrete edge (if \( c < c_{cr,N} \)) is considered by the factor \( A_{c,N}/A_{c,N}^0 \). During the tests, only the number of anchors in a row was varied, whilst the concrete edge distance and the anchor spacing were kept constant. The tested anchor group configurations are shown in Figure 8.

Firstly, the actual projected areas for all above cases were calculated according to pr EN 1992-4. Then, the projected areas of the groups near to edge and far from edge were compared (\( A_{c,N}^{2,near}/A_{c,N}^{2, far} = 0.88 \) and \( A_{c,N}^{3,near}/A_{c,N}^{3, far} = 0.91 \)). The factor \( \psi_{s,N} \) according to pr EN 1992-4 was the same for both the 1x2 and 1x3 configurations (\( \psi_{s,N} = 0.88 \)) because during the calculation, only the edge distance of the closest anchor is considered. Subsequently, the ultimate load of the groups near to edge and far from edge could be calculated according to Equation 4.

\[
\psi_{s,N} = 0.7 + 0.3 \cdot \frac{c}{c_{cr,N}} \leq 1 \quad (3)
\]

\[
N_{Rk,c} = N_{Rk,c}^0 \cdot \frac{A_{c,N}}{A_{c,N}^0} \cdot \psi_{s,N} \quad (4)
\]

where \( c \): edge distance of the closest anchor perpendicular to the edge; \( c_{cr,N} \): characteristic edge distance for ensuring the transmission of the characteristic resistance of a single break-out. The group tests were carried out far from the edge and near to the edge on both configurations and subsequently, the relation of the ultimate loads of 1x2 and 1x3 groups near to the edge and far from the edge were calculated as follows: \( N_{u,m,test}^{3,near}/N_{u,m,test}^{2,near} = 1.44 \) and \( N_{u,m,test}^{3, far}/N_{u,m,test}^{2, far} = 1.30 \). If we assume that the same actual projected areas are valid for both the calculations and for the tests according to prEN 1992-4, the change in the influence of \( \psi_{s,N} \) can be determined independently from \( A_{c,N}/A_{c,N}^0 \) by Equation 5 and 6.

\[
\frac{N_{u,m,EN}^{2,near}}{N_{u,m,EN}^{3,near}} = \frac{A_{c,N}^{2,near}}{A_{c,N}^{3,near}} \cdot \frac{N_{u,m,EN}^{2, far}}{N_{u,m,EN}^{3, far}} \cdot \psi_{s,N,EN}^2 \quad (5)
\]

\[\Delta \psi_{EN} = 1.0\]
According to prEN 1992-4, the factor $\psi_{s,N,EN} = \psi_{3,N,EN}^2 = 0.88$ and therefore, $\psi_{s,N,EN}^2 = 0.88$. The test results showed that the negative influence of the concrete edge ($\psi_{s,N}$) on the concrete cone capacity is less pronounced when having three instead of two anchors in a row $\psi_{s,N,EN}^2 / \psi_{s,N,EN}^3 = 0.92$ ($\psi_{s,N,EN}^2 = 0.88; \psi_{s,N,EN}^3 = 0.95$) (Figure 9a). Therefore, the factor $\psi_{s,N}$, which accounts for the influence of the concrete edge should be multiplied by a correction factor. The proposed correction factor is dependent on the number of anchors in a row and the product of $\Delta \psi_{s,N,corr}$ and $\psi_{s,N}$ must not exceed 1.0 ($\Delta \psi_{s,N,corr} \cdot \psi_{s,N} \leq 1$) (Figure 9b). The proposed correction factor can be expressed by Equation 7.

$$\Delta \psi_{s,N,corr} = 0.84 + 0.0795 \cdot n \quad \text{with the limitation:} \quad (\Delta \psi_{s,N,corr} \cdot \psi_{s,N} \leq 1)$$

where $n$: number of anchors in a row perpendicular to the edge. Note that the prEN 1992-4 covers only the application of groups with not more than 3 anchors in a row and regarding this, the tests were carried out only on 1x2 and 1x3 anchor groups. However, according to Figure 9b, it is assumed that the correction factor increases further with increasing number of anchors in a row. Further experiments should be performed on diverse anchor groups near to concrete edge with different edge distances to investigate the influence of the concrete edge and consider it adequately.

4.5 Influence of concrete edge in the case of eccentric loading

The concrete cone resistance of eccentrically loaded anchor groups on concrete edge was investigated on two different configurations and with varying whether the eccentricity (of external forces) is measured towards the edge or away from the edge. The test results showed that the capacity of an anchor group on the concrete edge is lower, when the eccentricity is measured towards the edge (Figure 5 and Figure 6). In the case of tests on 1x3 groups, when the point of load application was away from edge (G23, -e), the capacity of the group was 26% higher, than if it was loaded on the side near to the edge (G22, +e). According to prEN 1992-4, only the loading case:
eccentricity towards the edge is considered, and therefore is conservative for cases, where the eccentricity is measured away from edge (see Figure 5). This effect can also be seen in the case of 2x2 anchor groups near to concrete edge, loaded 2x eccentrically in tension (G43, -e). In this case, the group tests were carried out only in the way that the eccentricity was measured away from edge, therefore the reduction due to the eccentricity was compared with centric test results on edge. The concrete cone capacity decreased by 21% in comparison with centric tests on edge (G42). According to prEN 1992-4 the reduction of the concrete cone capacity was 41% due to the eccentricity (Figure 6). The reason for the deviations between the results according to the tests and prEN 1992-4 can be explained by the factor $\psi_{ec,N}$. When anchor groups are positioned on the concrete edge and are loaded eccentrically in tension, the characteristic resistance of the group is calculated according to Equation 5. The influence of the edge distance is considered as described in Section 4.4 and the influence of the eccentric loading is taken into account by the factor $\psi_{ec,N}$ (see 4.2, Equation 1). Hence, $e_N$ is a distance, an absolute value without defining whether the load application is closer to the anchors near to edge or closer to the anchors far from the edge. So, the factor $\psi_{ec,N}$ does not take into consideration the “direction” of the eccentricity of the tension loading.

$$N_{Rk,e} = N_{Rk,c}^0 \cdot \frac{A_{c,N}^0}{A_{c,N}} \cdot \psi_{s,N} \cdot \psi_{ec,N}$$  \hspace{1cm} (5)

In Figure 10, an anchor group configuration is shown, where all anchors of the group are tensioned. In this case, according to Equation 5, Load Case 1 (LC1) and Load Case 2 (LC2) deliver the same result for the characteristic resistance for concrete cone failure. In reality, however, the anchor near to concrete edge has lower resistance than the anchor which is farther from the edge. Consequently, and as the tests results highlighted, the resistance of the group cannot be independent of the “direction” of the loading eccentricity (LC1 or LC2) and should be therefore taken into consideration. The concrete cone resistance of the group in the case of LC2 must be higher than in the case of LC1 because in general, anchors which are closer to the point of load application have to take up higher forces than those which are farther. However, if the anchors close to the point of load application are near to concrete edge, then the resistance of these anchors is limited by the edge. According to these considerations, in the calculation of the concrete cone resistance of eccentrically loaded anchor groups on concrete edge, the prEN 1992-4 should distinguish whether the eccentricity is measured towards the edge or away from the edge.
4.6 Influence of cracked concrete

To investigate the influence of the different concrete conditions within a group, centric and eccentric tension tests were carried out on anchor groups in non-cracked and cracked concrete (according to Table 3). The test results showed that assuming cracked concrete for all anchors of the group leads to conservative results in most of the tested cases (up to 35% reserve – see Table 4). However, other tests (G63) showed that in certain unfavorable cases when only some anchors of the group are in cracks, the ultimate capacity of the group can be smaller than expected according to the calculations with assuming cracked condition for the whole group. The load distribution within a group depends on the actual crack pattern in the concrete component. The anchors positioned in cracks are less stiff and they exhibit smaller ultimate loads than the anchors in non-cracked part of the concrete. In this case, the anchor plate tends to rotate around due to the change of the lever arm of the internal forces. Consequently, the ultimate resistance of the group is reached before the anchor with the highest resistance and stiffness would reach its ultimate resistance. The results in Table 4 are shown as values relative to the case in which all anchors are positioned in cracks (which behave equally if loaded centrically). Based on these results, the ultimate concrete cone capacity of anchor groups is influenced by the number of anchors in cracks. In the case of eccentric loading, the ultimate capacity of the group is depending on the “direction” of the eccentricity as well. When the point of the load application is close to the anchor, which is installed in crack (G65), the measured ultimate concrete cone capacity was 25% less than if the loading was on the opposite side (G66). This can be explained by the fact that if anchors close to the point of load application are in cracks, then the resistance of these anchors is limited due to the disturbed stress condition in the concrete and the transfer of tension loads in this area is reduced.

5 Conclusion

The results of this study showed that in certain cases the prenormative standard prEN 1992-4 does not give reasonably accurate results for the permitted anchor group configurations. In this study, the mean ultimate concrete cone resistance of anchor groups was investigated. Pull-out tests in cracked and non-cracked concrete were performed on anchor groups with diverse configurations covered by prEN 1992-4 with particular interest on the influence of eccentric loading, loading on the concrete edge and cracked concrete condition. The mean ultimate loads were compared to the results of the calculations according to prEN 1992-4 to emphasize the limitations of the code. The comparison showed that the current code gives either conservative or under certain conditions non-conservative approximation for the ultimate concrete cone capacity of anchor groups for the investigated cases. The investigations on the influence of uniaxial eccentric loading on anchor groups far from edge showed that prEN 1992-4 gives conservative results for groups with 3 anchors in a row and the difference between the calculation and test results increases with increasing eccentricity (up to 21%).

The test results on the influence of the biaxial eccentric loading pointed out that due to the multiplication of the factors \( \psi_{ec,N,1} \) and \( \psi_{ec,N,2} \) according to prEN 1992-4, the calculated results for the concrete cone capacity of anchor groups are conservative (14% deviation between test results and calculation). The results of centric tension tests on anchor groups near to concrete edge showed that considering the influence of concrete edge by the factor \( \psi_{s,N} \) can lead to conservative results if anchor groups exhibit more than 2 anchors in a row. A correction factor was proposed to consider the...
influence of the number of anchors in a row. However, more tests are needed to consider the influence adequately. Furthermore, test results verified that the concrete cone resistance of eccentrically loaded groups near to concrete edge depends on the “direction” of the loading eccentricity and not only on the distance between the point of load application and the center of gravity of the tensioned fasteners (up to 26% higher loads when loading away from edge). Consequently, prEN 1992-4 should distinguish whether the eccentricity is measured towards the edge or away from the edge. Test results showed that assuming the cracked concrete condition is conservative for the most of cases. However, for certain unfavorable cases (when only some anchors of the group are in cracks) the ultimate concrete cone capacity of the group can be smaller, than expected according to prEN 1992-4 with assuming cracked condition for the group. A design concept should be developed that is applicable for the design of anchor groups when accounting for parameters i.e. arrangement of anchors, edge distance, eccentricity of loading, crack location and anchor plate stiffness.

6 Acknowledgement

The financial support received from the company fischerwerke GmbH & Co. KG is greatly acknowledged. The support and efforts of Dr. Joachim Schätzle, fischerwerke are greatly appreciated.

References:


5. DIN EN 12390-1:2012-12 Prüfung von Festbeton – Teil 1: Form, Maße und andere Anforderungen für Probekörper und Formen; Deutsche Fassung EN 12390-1:2012


CERTIFICATION AND APPROVALS
ACCEPTING NEW CONCRETE CONNECTION DEVELOPMENTS IN THE USA

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ABSTRACT

Product approvals in the USA as typically decided by the local authorities as no national authority exists. This paper discusses the history of authorizing concrete connections in buildings along various means for approving proprietary and unique connection methods not previously authorized in building codes and ordinances.

Building codes tend to be brief and succinct in the stated provisions, which is somewhat unfortunate when one would wishes to understand the intent and history of certain provisions in order to apply them to alternative materials. The model codes have made some improvements in providing commentary and references, but these items tend to be lack detail and are difficult to locate. This is significant as the codes obligate the local authorities to consider state of the technologies that have not yet made their way into the codes and new technologies initiated by manufacturers that reflect a significant investment in resources.

Available concrete connections have expanded significantly due to new technologies. In some cases the connection types from various producers have sufficient similarities to eventually enter the codes. Yet most products contain original content, and have unique designs, installation methods and performance characteristics. The justification for the performance characteristics consumes volumes of data. Designers need to gather sufficient knowledge on the products and then decide whether the product is acceptable and then seek approval from the code official. The dilemma of knowing what to ask for and deciding whether what is available is satisfactory exists.

While some large jurisdictions have procedures and staff dedicated to approving proprietary connections, most local authorities lack the time and expertise to dedicate to such effort. Research or evaluation reports have become a viable option for streamlining the approval process. Many countries employ a government sponsored agency for such a task, but in the USA, report issuers are often private concerns. In most cases the agencies issuing the reports operate according to ISO/IEC 170651 and use the codes and other normative documents to base the content. To assure continued conformance, the agencies arrange for follow-up inspections of the manufacturing facilities.
1 Introduction

Most people would probably take building products and materials for granted, believing that if they did not function as intended, they would not be used. The most obvious materials are those that are readily seen – roofing, exterior wall cladding, interior wall, ceiling and floor finishes, along with plumbing, electrical, and ventilation fixtures. There would be a realization of certain unseen elements, such as the foundation, and underlying structure of the walls, floors and roofs, which create usable space, protected from weather, for human activities or as repositories for storing items. Then there are the unseen products beyond the realization of many, but providing important function. Concrete connections are one such group of products. The International Building Code® (IBC)², currently the basis of most construction regulations in the USA and other countries, is guilty of this lack of realization when it comes to testing structural elements. While the connections may be implied by in situ testing, for prototype testing, the IBC discusses performance of the building element – wall, floor, or roof – without mentioning tests of connections.

While the IBC does either within its pages or by reference, provide information on using certain concrete connections, many are not addressed. A means for testing really is not within its scope either. Therefore, research reports, issued by model code agencies and other organizations provide the validated information for regulators, designers, builders, and owners to reference in verifying and selecting the connecting elements.

2 Developments in the U.S. Codes

One of the earliest mentions of connections in U.S. building codes pertains to anchors. Cast-in-place anchors were the first types to be described by the building codes, with anchorage using headed cast-in-place bolts first appearing in the 1943 edition of the Uniform Building Code™ (UBC)³. Here, permissible shear values for the anchor types were published, with tension capacities added in 1970. These values were reported in tabular form, with strengths predicated on bolt diameter and embedment.

In 1997, the UBC ceased publication and the International Building Code (IBC)¹ was published in 2000 as a replacement. The 2000 IBC contained the first mention of cast-in-place anchors for use in cracked and uncracked concrete and allowed for strength design, in addition to tabulated service or allowable loads taken from the IBC. In 2002, ACI 318⁴ added an Appendix D, with provisions similar to the IBC for cast-in-place anchors and significantly, adding methods for use of mechanical (expansion and undercut) anchors.

This move by ACI was significant as ACI 318 allowed for proprietary products to be code-complying by relying on proprietary testing. Unlike cast-in-anchors, where the justification is largely available on publically available testing and research, ACI permitted the actual testing on mechanical anchors to be kept from the public, provided the testing was assessed by an Independent Testing and Evaluation Agency (ITEA), who provided a summary of information to be used by regulators, designers, and builders.

While the evaluation report was intended to summarize a significant amount of information, it was a case of too much at once. First, the Appendix D represented a fairly significant increase in anchor design complexity, particularly since earlier codes provided the information in easy to follow tables.
Later, in 2011, ACI 318 added adhesive anchors. Appendix D presented an extensive set of design information to independently assess failure modes – steel failure, concrete breakout, anchor pullout, and anchor pryout – that would be computed for a selected installation condition and the least of which would be the design value. The set-up of the evaluation report did not relate clearly to the Appendix D. Soon computer software made the task of calculating anchor strengths easier. Manufacturers found that most of their accumulated research was unusable and retesting was needed. The resistance to Appendix D continues to this day, even though ACI now makes the anchor provisions part of the code, in Chapter 17.

The codes also recognize power-driven fasteners, which resemble nails and are produced from hardened steel. AISI S100\(^5\) includes provisions for power-driven fastener placement into steel and concrete. The specification does not provide for a clear discussion on pullout from concrete, however, limiting is usefulness as a design reference.

Other types of available fasteners include screw anchors, anchor channels, and specialty cast-in-place devices suited for connecting steel and wood light frame to concrete. These connectors do not currently enjoy inclusion into codes; usable information relies on criteria and research reports. Table 1 summarizes the connectors and the relevant codes, standards, and supplemental references for each.

### 3 Alternative Materials

The IBC Section 104.11 discussing alternative materials, granting the code official broad authority to approve these substitutions for compliant materials. Unlike most other countries, the United States federal government does not mandate a national set of codes, but rather defer to local rule, which today results in the states making a determination on code use. Or the state can itself defer to local cities and other jurisdictions to make a determination.

Most states and local jurisdictions could ill afford to develop, educate and train their employees on code writing. In the early 20\(^{th}\) century, code associations were formed, who eventually developed codes that were adopted with minor to significant revisions by the political entities. Aside from publishing codes, these associations took on the responsibilities of providing educational services and research reports.

#### 3.1 Parallel History

One of the earliest building codes with wide use in the USA was the 1927 Uniform Building Code\(^3\). The 1927 UBC was 46 Chapters, 256 pages in length. Specific references to building materials were included, along with testing and specifications standards by the American Society of Testing and Materials. Specific mention of proprietary materials was not included. However, Section 302 allowed for alternate materials and types of construction. Here the Building Inspector could accept the alternate provided one deliver proof in support of the claims made and request approval. Not much is given guidance was actually detailed in the UBC for determineing acceptance. If the building inspector denied approval, the applicant could appeal the decision of a Board of examiners and appeals. This language essentially remains to this day in the IBC. The codes assume all involved had immense knowledge and experience and will arrive at a proper decision. If the building inspector could not render an approval, and no further support was available, a board could step in and arrive at a final decision.
So, building inspectors could either take on the task and review the supporting materials before rendering approval or accomplish the same result with less effort. As one can imagine, this could be daunting, since it is one thing to know the code content, another to understand the reasoning and background information, and yet another to use this accumulated knowledge to determine the viability of an alternate material. In some cases, larger jurisdictions were up to the task. Should the alternate not be alternate at all, but something totally new, such as using adhesives as a structural material, then one would have to allow rational judgements for determining acceptance under the codes. There is also an implied objective to do the same thing consistently. So, if Company B came in seeking approval, knowing that Company A, had already, both companies should have the comfort of knowing that the bar was set at the same level for each. If none existed, often the case for new products, the code official should apply the rules consistently but the premise is that each permit is treated differently, and different individuals would likely interpret the code based primarily on the information available.

Early on, manufacturers found this to be true and petitioned the jurisdictions to do something about it. Larger building departments formed departments to determine acceptability of products within its jurisdiction, but these were few and far between. Around 1931, one of the early code agencies, the Pacific Coast Building Officials, formed committees to review manufacturers’ data and issue reports summarizing their conclusions for interested parties. This was a great service for all parties interested in using product and needing some assurance that the right questions were asked and the right data

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Table 1: Approvals of Concrete Connections
was submitted to make sure the full intent of the code is met. Eventually the committees found
themselves unable to devote resources needed to meet demand so it was necessary to bring on full
time and part time staff to keep up with the flow of work.

4 Research Reports

Research reports provide a means by which the code official can rely on a third party to review test
and other data to convey information to permit use of a product as meeting principles and specific
items in the IBC. The International Building Code Section 104.11.1 refers to research reports as a
source of information on alternative building materials. This reference is fairly new, first appearing
in the 2003 IBC. It is also brief, reading as follows “Supporting data, where necessary to assist in
the approval of materials or assemblies not specifically provided for in this code, shall consist of
valid research reports from approved sources.” The IBC also does not define what a research report
should be. So this obligation would be left to the approved source shaped by the code official’s
needs. Approved source is defined as “An independent person, firm or corporation, approved by the
building official, who is competent and experienced in the application of engineering principles to
materials, methods or systems analyses.” By this definition then, anyone who may be knowledgeable
in engineering would conform. As written, it could narrowly mean an engineering report issued by a
person found by the code official to be competent in the subject.

In retrospect, early attempts at research reports focused on available information from each producer
of the product, as the issuing organization made efforts to demand such information conform to the
requirements of the codes. As late as 2002, model code agencies issuing reports did not require any
inspections to verify ongoing production of products conformed to the specifics given in the research
report. Rather the norm was that the manufacturer was on the honor system and would inform the
research report issuer whether the product had changed and determine whether retesting would be
needed or not.

Currently, most research reports are developed by organizations accredited to ISO/IEC 17065 as
certification bodies. Accreditation is provided by accreditation bodies operating under mutual
recognition associations such the International Laboratory Accreditation Council (ILAC) and the
International Accreditation Forum (IAF). These organizations tend to be model code agencies,
standards development organizations, testing laboratories, and inspection agencies. One of the
mandates of ISO/17065, which is not implicit in the IBC, is ongoing inspection of the products being
certified in the report. Inspections are intended to provide a form of guarantee by the report issuer
that the product will perform as indicated in the research report. Tests and engineering services are
fairly expensive but only provide results for the products being investigated at that point in time. By
inspections, the certification body can verify the products being reported will continue to be expected
to perform as well as

4.1 Research Reports vs. Evaluation Reports

For the purposes of this paper, research reports are documents issued by an entity independent of the
manufacturer that documents conformance to the IBC and other building codes. These reports
typically address all relevant aspects of the codes. Aspects include structural use, installation, and
durability against weather exposure, fire resistance, and inspections for manufacture and field
installation. Evaluation reports are documents specified by ACI 355.2\(^6\), ACI 355.4\(^7\) and acceptance criteria that describe details for design and installation for use under ACI 318. These evaluation reports are usually done by one of the testing agencies. In some cases the evaluation report may mention the relevant ICC acceptance criteria. Table 1 notes availability of research reports and evaluation reports for each connector type.

5 General Notes on Anchors under ACI 318

The maximum diameter is 4 inches, which coincides with ASTM F1554\(^8\) (ACI 318 17.3.2.2). No minimum diameter is stated by ACI 318, though ASTM F1554 relies on a ½ inch (12.7 mm) diameter as a minimum. Post-installed anchors (mechanical, adhesive) have been qualified at diameters as small as 1/4 inch (6.4 mm). With respect to anchor steel strengths, ACI 318 offers no direct statements, though ASTM F1554 gives a minimum expected tensile strength of 58 ksi (400 MPa). The maximum permitted by the strength calculation, however, is the lesser of 1.9 fy or 125 ksi (861.8 MPa). ACI 318 does not explain about stainless steels or zinc-coated carbon steels. Zinc-coated carbon steels are covered in ASTM F1554 while stainless products can be selected by reliance on ASTM A193\(^9\) and ASTM F593\(^10\). Section 1704.5 of the International Building Code requires special inspection during installation. ACI 318 is somewhat vague on inspection requirements, though adhesive types are discussed in fair detail.

5.1 Cast-in Bolts

The scope of discussion on cast-in anchors here focuses on circular bolts with a bearing element in the form of a manufactured head or nut and washer at the embedded end. Bolts with a bent end into an “L” or “J” shaped hook also provide a bearing element and are usable here. Table 2 summarizes data to be considered in order to comply with the IBC and ACI 318 since these anchor types do not possess research reports.

Design requirements are provided in ACI 318. The IBC Section 1905.1.8 also allows an alternative design method for cast-in bolts when attaching These cast-in devices must be proven to develop apparent pullout strength at least 1.4\(N_p\) (ACI 318 17.1.3) where \(N_p\) is the calculated pullout capacity based on the geometry of the bearing element. ACI 318 does not elaborate on how the requisite pullout strength would be determined, but most likely a set of tests of the anchor set in concrete would be appropriate. ACI 318 also does not specify testing methods either. One test procedures often used is ASTM E488\(^11\). ACI 318 also does not provide guidance on the number of test specimens for the basis of comparison. Would the basis be the mean result, the least result, or the 5 percent fractile value? The design professional and code authority would have to reach agreement here. The commentary to ACI 318 indicates that headed bolts complying with ASME B1.1\(^12\), B18.2.1\(^13\), or B18.2.6\(^14\) would acceptable by calculation, so no tests are necessary. ASME B18.2.6 is titled Fasteners for Use in Structural Applications and encompasses both dimensional and structural properties for heavy hex-head bolts. ASME B18.2.1 only addresses dimensional requirements for square and hex-head bolts, while ASMRR B2.1 discusses screw threads. Neither ASME B2.1 nor B18.2.1 addresses structural properties. Structural Properties are based on ASTM F1554, which is available in Grades 36, 55, and 105. If the design professional wished to check pullout values, the AISC Design Guide\(^15\) provides a table using Eq. 17.4.3.4 of ACI 318.
Table 2: Checklist for Cast-In Anchors

<table>
<thead>
<tr>
<th>Property</th>
<th>Headed</th>
<th>Hooked</th>
<th>Threaded Nut and Washer</th>
<th>Stud</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dimensions</strong></td>
<td>ASME B1.1, B18.2.1, B18.2.6</td>
<td>ASME B1.1</td>
<td>ASME B1.1</td>
<td>AWS D1.1 Clause 7</td>
</tr>
<tr>
<td><strong>Materials</strong></td>
<td>ASTM F1554 (carbon)</td>
<td>ASTM F1554 (carbon)</td>
<td>ASTM F1554, ASTM A193 B7 (carbon); B8 (stainless)</td>
<td>AWS D1.1 Clause 7</td>
</tr>
<tr>
<td><strong>Size</strong></td>
<td>ACI 318 17.3.2.2</td>
<td>ACI 318 17.3.2.2</td>
<td>ACI 318 17.3.2.2</td>
<td>ACI 318 17.3.2.2</td>
</tr>
<tr>
<td><strong>Ductile/Brittle Steel</strong></td>
<td>ACI 318 2.3</td>
<td>ACI 318 2.3</td>
<td>ACI 318 2.3</td>
<td>ACI 318 2.3</td>
</tr>
<tr>
<td><strong>Pullout Data</strong></td>
<td>Note *</td>
<td>SP 292-2</td>
<td>--</td>
<td>79-77</td>
</tr>
</tbody>
</table>

*Headed bolts conforming to ASME B1.1, B18.2.1, or B18.2.6 can be accepted without pullout data.

Headed bolts can be forged onto the end of the bars to shapes conforming to ASME B18.2.1 or B18.2.6. For larger diameters, it may not be possible to obtain a forged head, so the option to use nuts and either plates or washers to form the bearing area may need to be pursued. Substituting a forged head bolt with threaded rod and nut may result in a lesser strength connection since the stress area of the threaded section near the nut is less than the full cross section provided by a forged head bolt.

Though not mentioned in ACI 318, headed studs conforming to AWS D1.1\textsuperscript{16} may be a suitable alternative to headed bolts. These anchors are not threaded and are suitable for anchorage for weld plates. The head dimensions for these connectors suggest the head width to thickness ratio is less stiff than what is specified for heads in ASME B18.2.1 and B18.2.6. Some research on tensile strength (79-77)\textsuperscript{17} suggests the studs are superior to Grade 36 bolts.

For headed bolts, ACI 318 provides no guidance on pullout. A recent publication (296-2)\textsuperscript{18} suggests pull-out strengths for L-shaped hooked bars could be increased beyond that currently in ACI 318.

Hooked bars likewise are produced from steel conforming to ASTM F1554. The standard specifies that the bend section zone not be so deformed that cracking occurs under 10 x magnifications. Also, the cross-sectional area must be no less than 90 percent of the straight section. Bars that exhibit a notch were likely bent at a small radius and should be avoided since the notch is a stress riser and the bend is where the stress are concentrated. Bends in the threaded zone should be avoided.
5.2 Expansion Anchors
Expansion anchors transfer loads in friction by setting a component that wedges against the concrete hole. The holes need to be relatively precise and clean to assure proper wedge action. Torque set anchors need to be set by the using the manufacturers recommended torque values and not the installer’s “turn-of-the-nut” method. In some cases the hole may intersect the reinforcing steel. The installer may try to elongate the hole to avoid the steel, but this will likely reduce the anchor capacity. Design data are based on ACI 318, and qualification is taken from ACI 355.2 and AC19319. Information on design and installation can be found in a research report or an evaluation report.

5.3 Undercut Anchors
Undercut anchors transfer loads in bearing by on enlarged portion of the hole at the end into which the anchor expands. These fasteners are grouped separately in ACI 318 but qualification requirements in ACI 355.2 and AC193 do not distinguish undercuts from expansion type devices for the most part. These anchors are more suitable for dynamic loads than other post-installed types. Design data are based on ACI 318; information on design and installation can be found in a research report or an evaluation report.

5.4 Adhesive Anchors
Adhesive anchors transfer loads by a mechanically bonding a steel rod or bar to the concrete hole using an organic adhesive. The holes do not need to be as precise as expansion anchors but need to be cleaned according to the manufacturer’s instructions using a sequence of brushing, and blowing or vacuuming to remove debris. Adhesives are sensitive to temperature effects both during installation and in service. Installations into horizontal and overhead orientations need to observe manufacturers instructions for placement and retention until cure is complete. For these reasons both the IBC and ACI 318 prescribe special inspection, either continuous or periodic. The conditions stipulating special inspection are examined in both the research report and the evaluation report. The research report also provides conditions where the adhesive anchor could be used in fire-resistive construction. There are fire test methods in ASTM E151220 but most research reports have not provided this information. These products are qualified in accordance with ACI 355.4 and AC30821. Design details are given in ACI 318 while information on design and installation can be found in a research report or an evaluation report.

5.5 Screw Anchors
Screw anchors mechanically bond to concrete along the entire length. Neither ACI 318 nor ACI 355.2 currently includes screw anchors. AC193 is the most specific resource for the qualification and design of these anchors. The anchors are produced from hardened steel to thread into the concrete. This hardened steel is subject to brittle failure, which is examined by AC193. Users are cautioned to investigate this failure mode if choosing an anchor without a research report. Design provisions are taken from ACI 318, except pullout resistance needs to be taken from test results. Steel tension and shear values are typically tabulated as these fasteners often have unique dimensions.
5.6 Anchor Channels
Anchor channels transfer loads by way of cast-in elements bearing to concrete. The cast-in element is in turn connected to the channel, which receives a channel bolt. AC23222 is the main reference for qualifying the overall design and installation of these devices. The channel bolts can be set along the channel length allowing flexibility in positioning a connection. One challenging aspect of anchor channels is the slotted geometry does not allow for superior performance when loaded in shear along the channel axis. In particular, solutions for dynamic loads such as seismic sources have been limited. Design provisions for each element are determined separately to arrive at a connection strength. Research reports on these connectors are typically quite comprehensive in addressing pertinent design considerations.

5.7 Headed Specialty Cast-in Inserts
Headed specialty cast-in inserts transfer loads to concrete in bearing similar to a headed anchor bolt. These connectors typically have an internally threaded element to receive bolts or rods. The embedment into the concrete is around 1½ to 2 inches (38 to 51 mm) and may be placed through the underside of concrete-filled steel deck panels. As such the resulting capacities are appropriate for support of non-structural elements. Qualification of these inserts is not addressed in either the IBC or ACI 318, but AC44623 is available for this task. The anchorage design is according to ACI 318 though pullout often not limiting due to the shallow embedment. Research reports provide extensive detail on steel strengths, due to the unique configurations available.

6 Other Concrete Connections

6.1 Power Driven Fasteners
These fasteners resemble nails in appearance and function by transferring loads by friction to the concrete. Formed from hardened steel, power driven fasteners are also suitable connecting attachments to steel. Unlike anchors, these fasteners are not suited for installation where cracks are present or expected by design. Design details are given in AISI S100, though currently most research reports are qualified by a series of tension and shear tests as stipulated by AC7024.

6.2 Specialty Inserts
Other connectors available are not entirely subject to design according to ACI 318 and structural use is largely supported by tests in concrete. Research reports are available for many of these types. These devices include:

1) Sill plate strap anchors, which attach light frame wood or cold-formed steel members to concrete. These devices are formed from cold-formed steel and need not have deep embedments since overall capacity would be limited by connection to the attached wood or steel member.

2) Deformed bar anchors are deformed bar or wire serving as anchorage. These products serve as a dowel or may be welded to steel plates.
3) Specialty bolts are bent into unique configurations and cast into concrete. As such, the bolts can be embedded fairly deep into concrete. These anchors currently are used for tension loadings only.

7 Conclusions

The Building Codes in the U.S.A. have long acknowledged the need for regulators to allow for alternative materials. Yet the codes do not provide much beyond general provisions to assure full consideration is given toward determining acceptability. With respect to concrete connections, several types including cast-in bolts, expansion anchors, undercut anchors, and adhesive anchors are permitted by ACI 318. ACI 318, however, focuses on structural performance and does not completely guide users on other aspects such as fire exposure, durability, and inspection, which are under the scope of the International Building Code (IBC). Therefore specifiers, users, and regulators would benefit from mutually acceptable resources documenting these characteristics.

Research reports offer a comprehensive reference for assessing code compliance. The strength of available research reports is even greater for connections that are not described in the IBC or its reference standards at all, such screw anchors, anchor channels, and headed specialty cast-in inserts. Research reports on these products are in most cases based on criteria documents, which provide direction as to comprehensiveness and consistency among reports on similar products from different manufacturers.

Cast-in anchors currently lack support in the form of research reports and neither the IBC or ACI 318 reference requirements in sufficient detail. Information presented in this paper will provide guidance toward acquiring data that satisfies the codes.

Figure 1 provides an advisory on steps to consider in selecting a connector and gathering evidence to support its use under the IBC and supporting reference standards. As always, inclusion of the code official in the decision is vital when the project is regulated by localities.
Figure 1: Steps in Selecting a Suitable Connector.

References:

1. ISO/IEC 17065, Conformity assessment — Requirements for bodies certifying products, processes and services, International Organization for Standardization (ISO), 2012


4. ACI 318, Building Code Requirements for Structural Concrete, American Concrete Institute, 2014

5. AISI S100, North American Specification for the Design of Cold-Formed Steel Structural Members, American Iron and Steel Institute and CSA Group, 2012

6. ACI 355.2, Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary, American Concrete Institute, 2007
7. ACI 355.4, Qualification of Post-Installed Adhesive Anchors in Concrete, American Concrete Institute, 2011


12. ASME B1.1, Unified Inch Screw Threads, (UN and UNR Thread Form), American Society of Mechanical Engineers, 2003

13. ASME B18.2.1, Square, Hex, Heavy Hex, and Askew Head Bolts and Hex, Heavy Hex, Hex Flange, Lobed Head, and Lag Screws (Inch Series), American Society of Mechanical Engineers, 2012

14. ASME B18.2.6, Fasteners for Use in Structural Applications, American Society of Mechanical Engineers, 2010


24. AC70, Power-driven Fasteners Driven into Concrete, Steel and Masonry Elements, ICC Evaluation Service, 2016
ASSESSMENT OF PERFORMANCE, FREE TRADE AND USE OF FASTENING SYSTEMS WITHIN THE EUROPEAN ECONOMIC COMMUNITY (EEC)

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ABSTRACT

The construction sector is very important for the economy in Europe. With the entry into force of Regulation (EU) 305/2011 on 1 July 2013, the so-called EU Construction Products Regulation (CPR), the legislation on construction products has been amended.

This law aims to reduce the construction sector of competitive and existing trade barriers. The CPR distinguishes between European and national rights much more clearly than before.

The EU now regulates the requirements on construction products, the Member States regulate the requirements for their use as well as the necessary national security.

To this end, the separation of the performance assessment from the use is aimed at and the Member States are invited to transform the existing requirements for construction products, to requirements for buildings.

Characteristics relating to such basic requirements for construction works not yet covered by the harmonised European standards and European Assessment Documents (EAD) are often regulated nationally. This inevitably leads to a tense relationship between the EU and the Member States.

The presentation provides an overview about the state of the art of the recognized technical assessment documents for post-installed fasteners within the EEC, about the evidence of performance for use, and about the consequences of free trade with CE marked fastening systems for practice.

1 Introduction

According to the European Commission the construction sector in Europe provides more than 18 millions direct jobs and contributes to about 9% of the gross domestic product of the European Union.

One measurement to get a sustainable and competitive construction industry, the Regulation (EU) 305/20111, the so-called EU Construction Products Regulation (CPR), the legislation on construction products has been amended. In conjunction with other regulations (e.g. Accreditation and Market
Andreas Bucher and Peter Schillinger

Surveillance) it aims to reduce barriers to trade by Technical Harmonization, Technical Assessment and Harmonised rules on product declaration and CE marking.

2 The legal framework for Construction Products

The CPR\(^1\) provides the legal framework for Construction Products in the European Economic Area, i.e. in the 28 EU Member States as well as in Iceland, Liechtenstein and Norway.

The CPR\(^1\) makes a much clearer distinction between European and National Law. Implementing this regulation the focus of the European Commission is to technically harmonize construction products trying to separate their performance from the use, which is National Law. At the same time the Member States are invited to transform their national requirements on construction products to requirements on buildings respectively on construction works.

2.1 The Basic Requirements on Construction Works

In Annex II the CPR\(^1\) defines 7 Basic Requirements on Construction Works (BRCW), briefly summarized as below:

1. Mechanical resistance and stability
   No collapse of the works, no major deformations etc.

2. Safety in case of fire
   Behavior and resistance

3. Hygiene, health and the environment
   Emissions of dangerous substances into air, water, soil etc.

4. Safety and accessibility in use
   No unacceptable risks of accidents or damage in service or in operation
   Accessibility and use for disabled persons

5. Protection against noise

6. Energy economy and heat retention
   Construction works must be energy-efficient, using as little energy as possible

7. Sustainable use of natural resources
   Recyclability of the construction works

This list lets suggest that the construction sector and so construction products are affected by several policies, which leads to high complexity for all stakeholders.

Nevertheless, for post-installed fasteners – as connections between steel and concrete – the main focus in the past was on BRCW 1, BRCW 2 and BRCW 4 only.

Characteristics related to such basic requirements not yet covered by a harmonized Technical Specification (European standard or European Assessment Document), have been often regulated nationally. One example are requirements on BRCW 3 “Hygiene, health and the environment”.

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1. CPR: Construction Products Regulation.
However, once a construction product has been CE marked, a Member State may not impose any further requirements on it. This inevitably leads to a tense relationship between the EU and the Member States.

2.2 The main routes to the CE marking

To market a construction product with the CE marking in the European Economic Area, the CPR provides two main routes:

The first way has to be taken, if the construction product is covered by a harmonized European standard (hEN), meaning that the standard completely describes the intended use of the construction product. Then the manufacturer of the construction product is obliged to (let) assess the performance of his construction product according to the criteria of the harmonized standard, to establish the rules for the assessment and verification of the constancy of performance (AVCP), to draw up the Declaration of Performance (DoP) and to affix the CE mark to the construction product.

The second route is optional for a manufacturer and can be followed, if there is no harmonized standard for the construction product or a harmonized standard does not cover a construction product completely. The manufacturer may then apply for a European Technical Assessment (ETA) at a Technical Assessment Body (TAB) for assessing the performance of the essential characteristics for its intended use in relation to the relevant BRCWs. In the case an ETA has been issued for the construction product, the manufacturer then also has the obligation to meet the rules of the AVCP system, to draw up the DoP and to affix the CE marking to the construction product.

It is in the nature of things that the so called ETA route is preferably used for the CE marking of new and innovative construction products.

2.3 The Assessment and verification of constancy of performance

To ensure the performance of a CE-marked construction product over time (after the assessment), the CPR provides systems for the Assessment and Verification of Constancy of Performance (AVCP). The applicable AVCP-system relevant for a construction product is laid down in the applicable harmonized technical specification (hEN or EAD).

It is the responsibility of the manufacturer to establish a Factory Production Control (FPC) system that guaranties the performance of the product as specified in the corresponding DoP.

The Control Plan agreed between the TAB and the manufacturer is the key document for the supervising Notified Body. The registry of Notified Bodies, notified by Member States, to perform the third party tasks, can be found on the NANDO website (http://ec.europa.eu/growth/tools-databases/nando/index.cfm).

In Figure 1 the tasks of the manufacturer and the Notified Body are described for the AVCP-systems:
2.4 The Declaration of Performance

When a construction product is covered by a harmonised European standard (hEN) or it conforms to an ETA, which has been issued for it, the manufacturer shall draw up a Declaration of Performance (DoP) when such a product is placed on the market.

The DoP is the manufacturer's declaration required for the CE marking which must be provided with the construction product or via the website of the manufacturer to the dealer or user (in the language of the Member State where the construction product is used; in total, there are 24 different languages to be considered). The DoP have to be made available over a period of 10 years after the construction product has been placed on the market.

If an ETA has been issued for the construction product, the manufacturer is obligated (in accordance with CPR Article 6, (3) g) to include into the DoP all performance values of the characteristics given in the ETA. The format of the DoP is defined by a Commission Delegated Regulation.

In order to achieve the widest possible applicability of the construction product, manufacturers will have to (let) assess all characteristics, which are relevant for the intended use. Essential characteristics included in the hEN or the EAD, which have not been assessed, are to be declared in the DoP with "NPD" (= no performance determined).

To avoid an empty DoP, at least one essential characteristic have to be declared in the DoP by the manufacturer. Unfortunately, this rule is often interpreted as dangerous, because a manufacturer may declare only one essential characteristic. However, also in the past the user of a construction product could not rely on the fact only, that a construction product carried the CE mark. The user always had the task to check resp. verify, whether the construction product meets the relevant requirements. Now, the DoP enables the installer to check, whether the performance of the required characteristics is declared and meets the (national) requirements.
Finally, provided by the legislature, the DoP is therefore the central document with regard to legal aspects between the building owner, the installer of the construction product and the manufacturer (e.g. during the acceptance of the works: documentation as to whether the built-in construction product can be used for the application).

2.5 European Assessment Documents

A European Technical Assessment (ETA) is issued on the basis of a European Assessment Document (EAD) which, if there is no suitable EAD existing, is prepared using the defined procedure according to CPR Annex II in¹.

This system allows EOTA and manufacturers to create EADs as soon as the essential characteristic relevant for the declared use to be assessed has an influence on the performance of the construction works.

So the EAD contains the essential characteristics (relevant for the intended use in relation to BRCWs) and their test and assessment criteria (to determine their performances).

EADs are published by EOTA and made available on the EOTA website (www.eota.eu).

For fasteners for the use in concrete (product area 33 “Fixings”, intended use 0601) the following documents are available as basis for issuing ETAs:

Table 1: EADs of the (status 27.04.2017)

<table>
<thead>
<tr>
<th>EAD Number</th>
<th>EAD Title</th>
<th>OJEU</th>
</tr>
</thead>
<tbody>
<tr>
<td>330008-02-0601</td>
<td>Anchor channels</td>
<td>2016/C 248/06</td>
</tr>
<tr>
<td>330011-00-0601</td>
<td>Adjustable concrete screws</td>
<td>2015/C 226/05</td>
</tr>
<tr>
<td>330012-00-0601</td>
<td>Cast-in anchor with internal threaded socket</td>
<td>2016/C 172/03</td>
</tr>
<tr>
<td>330030-00-0601</td>
<td>Fastener of external wall claddings</td>
<td></td>
</tr>
<tr>
<td>330075-00-0601</td>
<td>Elevator Lifting device</td>
<td>2016/C 172/03</td>
</tr>
<tr>
<td>330083-00-0601</td>
<td>Power-actuated fastener for multiple use in concrete for non-structural applications</td>
<td>2015/C 378/02</td>
</tr>
<tr>
<td>(superseded)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>330083-01-0601</td>
<td>Power-actuated fastener for multiple use in concrete for non-structural applications</td>
<td>2017/C 118/04</td>
</tr>
<tr>
<td>330084-00-0601</td>
<td>Steel plate with cast-in anchor</td>
<td>2016/C 378/08</td>
</tr>
<tr>
<td>330087-00-0601</td>
<td>Systems for post-installed rebar connections with mortar</td>
<td></td>
</tr>
<tr>
<td>330232-00-0601</td>
<td>Mechanical fasteners for use in concrete</td>
<td>2016/C 459/08</td>
</tr>
<tr>
<td>(replaces ETAG 001-1, -2, -3, -4)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>331072-00-0601</td>
<td>Anchor devices for fastening personal fall protection systems to concrete structures</td>
<td></td>
</tr>
</tbody>
</table>
According to recital (18) of the CPR existing European Assessment Documents shall be used as basis for the development of harmonised standards, as soon as a sufficient level of technical and scientific expertise on all the relevant aspects is attained.

Considering that European standards are usually revised every 5 years, one can suppose, that the European Commission (after consultation of the Standing Committee on Construction) will develop new mandates for standardization based on EADs in the medium-term in 5 to 10 years.

2.6 The European Technical Assessment

The content of the European Technical Assessment (ETA) is defined by a Commission Implementing Regulation. The ETA is issued on the basis of an EAD and is seen as a "snapshot" of the performance of a construction product at a given time. As an EAD serves to a certain extent the preparation for a standard, the ETA has actually a foreseeable period of validity. For these reasons, a limitation on the validity of an ETA is not seen as required.

In the field of fastening systems, the ETA states the performance of the construction product in the installed state containing supplementary boundary conditions (e.g. minimum member thickness) to ensure the performance in the installed state. Additionally, the ETA contains product-specific performance values taking into account foreseeable deviations on site (e.g. installation safety factor). Such factors have been established successfully many years ago to get rid of expensive tests on site, which were used for the verification of a correct installation.

For fastening systems the product performance is closely connected with the design method and vice versa. So the ETA contains also the reference to the related design method(s).

Especially for innovative products in relation to BRCW 1 and BRCW 2 it is important, that the ETA contains the reference to the recommended design method furthermore. Otherwise, due to the novelty of the product, the market would lack the experience of how such a product could be designed and so it would be doomed without the proof to fulfil certain requirements. So the reference to appropriate design methods is necessary.

However, the ETA generally does not contain anymore the partial safety factors required for the design, because use and safety are subject to National Law. The recommended partial safety factors are given in the related document, where the design method is described. In general, the partial safety factors determined by the Member States are given in a national annex to such documents.

2.7 The advantages of CE marked construction products

“By affixing […] the CE marking, manufacturers indicate that they take responsibility for the conformity of the construction product with the declared performance as well as the compliance with all applicable requirements laid down in this Regulation and in other relevant Union harmonisation legislation providing for its affixing.” (see CPR, Article 8(2))

In practical terms, the CE marking of a construction product indicates that, it has been assessed (respectively tested) based on the applicable harmonised technical specification (hEN or EAD).
The performance of competing construction products (provided the applicable harmonized technical specification is the same) can be compared on this basis. Thus it secures a level playing field (not only for manufacturers, even for customers).

Furthermore, the CE marking ensures the constancy of the product performance on the basis of a supervision system, the so called AVCP system. Via the DoP the performances of the construction product can be compared with the requirements on the construction works in the territory of a Member State. Such a simple comparison evidences, whether a construction product can be used there.

Finally, the CE marking enables construction products to move across the EU, avoids new assessments and certifications and provides thus free trade within the European Economic Community.

3 Evidence of performance for use

The Member States are responsible for safety, health and other matters covered by the basic requirements for construction works on their territory. Therefore, as already mentioned above, the use of construction products and safety are subject to National Law. Therefore Member States in Europe define national rules that have to be taken into account when processing construction works.

So there are various requirements to be taken into account to fulfil the basic requirements for construction works applicable to the intended use of each construction product in the territory of each Member State, e. g. on

- the training and experience of people processing construction products (e. g. for post-installed reinforcing bars)
- the design method introduced by the building authority (e. g. Eurocode 2)
- the stability of buildings (e. g. in Germany: requirements on the building owner by the Laender (the federal states), e. g. the building code of Baden-Württemberg)
- the verification of no emissions of dangerous substances into air, water, soil etc.

Finally, the building owner has the obligation to fulfil the national requirements. In the case he is not himself suitable for the fulfilment of the obligations, the building owner can usually appoint suitable parties (e. g. an architect).

Nevertheless, at least in Germany, the building owner shall provide the necessary supporting documents and documentation relating to the construction products and the types of construction used. If construction products bearing the CE marking in accordance with the CPR are used, the Declaration of Performance must be provided.
4 Consequences of free trade with CE marked fastening systems for practice

Having in mind that not every possible essential characteristic for the intended use (minimum one) is to be declared, at first the DoP of the construction product in question has to be checked, whether the declared performance can meet the relevant requirements (e.g. requirements on BRCW 1 and BRCW 2).

![Declaration of Performance](image)

Figure 2: Example of DoP (first page only)
The example of a DoP (see Figure 2) shows, that no essential characteristic is declared with “NPD” and that the essential characteristics for the intended use both for “static and quasi-static actions” and “seismic actions” in relation to mechanical resistance and stability (BRCW 1) and to safety in case of fire (BRCW 2) are given. The performance values can be found in the annex of the document.

It is easy to understand, that, if in a DoP the abbreviation ”NPD” can be found, caution is advised, because this characteristic may be essential for the application!

The DoP should be archived and provided as documentation for the acceptance of the works, whether the built-in construction product can be used for the application.

For post-installed fasteners then usually a design calculation is required for the verification of the product performance in relation to BRCW 1 resp. BRCW 2 taking into account the conditions on site. Only with this design calculation it is possible to proof that the performance of the fastening is sufficient and the requirements can be fulfilled.

Therefore, if the already verified fastener is to be exchanged against another product, a new design calculation is to be carried out – otherwise the building owner could not fulfil his obligations.

In the event of such an exchange serious manufacturers usually point to the need for a new design calculation and offer corresponding services.

5 Conclusion

During the development of harmonized European Standards or European Assessment Documents the separation of European and National Law has to be taken into account.

To be able to fulfil national requirements in relation to the BRCWs 3, 5, 6 and 7, at first the national requirements should be determined. Then these requirements can be transferred and harmonized on the European level to avoid incomplete harmonized European standards. All stakeholders are invited to participate actively in the development of such harmonised standards.

For the development of EADs resp. for getting an ETA including the performance for the required essential characteristics, it is important, that the installers and the manufacturers are in close contact. Then the manufacturer can apply for the ETA, which can serve to fulfil the national requirements and so the obligations of the installer.

References:


**Used abbreviations:**

EU – European Union  
CPR – Construction Products Regulation¹  
EAD – European Assessment Document  
DoP – Declaration of Performance  
ETA – European Technical Assessment  
hEN – harmonized European Standard  
AVCP – Assessment and Verification of Constancy of Performance  
TAB – Technical Assessment Body  
FPC – Factory Production Control  
NPD – No Performance Determined  
EEC – European Economic Community
ASSURING QUALITY ADHESIVE ANCHOR INSTALLATIONS
IN THE AMERICAS

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ABSTRACT

Anchorage to concrete design provisions were first incorporated into the ACI 318 Code in 20021 as Appendix D. In 2011, the design provisions for adhesively bonded anchors were added to the ACI 318-11 Building Code Requirements for Structural Concrete.2 ACI took a proactive position in response to the recommendations/criticisms from the National Transportation Safety Board (NTSB) resulting from an adhesive anchorage failure in the “Big Dig” tunnel accident in Boston, Massachusetts. Additionally, Adhesive Anchor Installer Certification was made a requirement in ACI 318-11 Code for anchorages with sustained tension loading and installation orientations from horizontal to vertically upward (overhead).

This paper provides an introduction and background to the adhesive anchor certification requirements incorporated into ACI 318 Code (now Chapter 17 in ACI 318-143). The ACI 318 code committee mandated a certification program for adhesive anchors. The program was fast-tracked with the aid of a certification development consultant, which worked with ACI Committee 601A - Adhesive Anchor Installer. ACI assembled a group of Subject Matter Experts (SMEs) knowledgeable about adhesive anchors, who identified the criteria and developed examination protocols that installer candidates must follow to obtain certification as ACI Adhesive Anchor Installer (AAI). The result of this fast-tracked effort is an operational certification program with both written and performance examination requirements that has been accepted by the industry and specified by code.

1 Introduction

In July 2006, tragedy struck a newlywed couple traveling on the Interstate 90 connector tunnel in Boston, MA, when precast concrete ceiling panels detached from the tunnel ceiling and crushed their car (Fig.1). The impact killed the wife and injured the husband. This unfortunate tragedy lead to immediate investigations into the cause(s) of the failure.

Findings in the National Transportation Safety Board (NTSB) Highway Accident Report – Ceiling Collapse in the Interstate 90 Connector Tunnel, Boston Massachusetts,4 released July 2007, provided recommendations as well as considerations to prevent future failures where life safety is of concern.
This report motivated many within the construction and anchoring industry to evaluate potential improvements and needed actions to further educate those using adhesive anchor systems.

Fig. 1: Twenty-six (26) tons of precast concrete panels release from ceiling and crush car.

The American Concrete Institute (ACI) was proactive in its desire to address some of these issues by implementing changes to the following codes and standards; ACI 318-11 *Building Code Requirements for Structural Concrete* and ACI 355.4-11 *Qualifications of Post Installed Adhesive Anchors in Concrete* specifications. In addition, ACI developed a certification program for Adhesive Anchor Installers, online educational programs, and hosted symposiums on anchorage to concrete. This paper focuses on the development and implementation of the Adhesive Anchor Installer certification program.

2 NTSB Investigation into Big Dig Failure

Conclusions in the NTSB report indicate the primary cause of the ceiling failure in Boston was related to the creep characteristics of the adhesive. The adhesive formulation used on this project was not approved for sustaining long-term loads. Displacement (movement) of the anchorage system was observed in a number of locations prior to the failure (see Figure 2). However, contractors, inspectors, manufacturers, and the Department of Transportation (DOT) were all learning “on the job” in their efforts to resolve this issue and avoid its recurrence. The NTSB report also indicated that a general lack of knowledge related to creep issues with adhesive anchors was apparent and the industry needed to take immediate action to understand and measure this important property.

Fig. 2: Photograph of hanger plates with displaced anchors adjacent to the failure area.
Another finding in the NTSB report was related to difficulties associated with overhead installations – Finding 17 states; “Installing adhesive anchors in overhead applications appears, by the nature of the task, to introduce voids into the adhesive that can reduce the ultimate load capacity of the anchor and thus the overall reliability of the anchoring system.”

In fact, many of the installed anchors examined from the scene of the collapse exhibited deficiencies with the installation. Of the 20 failed anchors, 19 were found to have void areas from 1 to 40% over their embedment length. The anchor used was a 5/8-inch (16mm) diameter threaded stainless steel rod (316 SS) with a required 5-inch (127mm) embedment depth. A total of 85 anchors were removed and examined from the tunnel. Twenty of the 85 anchors came from the collapse area and 65 were taken from outside the failure area. Sixty-two (62) of these 65 anchors were found to have void areas around the threaded anchor ranging from 1 to 70% of the total embedment area (see Figure 3). Forty (40) of these anchors had displaced from the tunnel roof before removal; reported displacements ranged from 0.625 to 2.65 inches (16 to 67 mm). It appears that installation practices could have contributed to the failure. If an anchor is installed to fasten a hanger plate and 40% of its embedment depth not fully engaged with adhesive, this would reduce the maximum load capacity of that anchor.

Fig 3: Anchors from the failure area exhibiting installation deficiencies.

Losing one anchor of many may not be critical, but the load has to redistribute and would transfer additional load on the adjoining anchors as well as the next hanger-plate installation. Considering that the anchors in the next hanger plate may also have installation deficiencies, this compounds the issue. Now add that the adhesive had poor creep properties…. failure.

The NTSB recommendations to the American Concrete Institute were: Use your building codes, forums, educational materials, and publications to inform design and construction agencies of the potential for gradual deformation (creep) in anchor adhesives and to make them aware of the possible risks associated with using adhesive anchors in concrete under sustained tensile-load applications. (H-07-32)

2.1 ACI Committee 318 Action

Members of the ACI 318 building code committee want assurances that those installations with sustained long-term loads would not be problematic in the future. The code committee first took the position that adhesive anchors not be allowed in the code in an overhead, sustained-load condition.
Extensive code committee discussions lead to agreement that ACI would develop a certification program for Adhesive Anchor Installers. If ACI developed a program accepted by the industry, the ACI 318 committee members were willing to write code language allowing adhesive anchors, however, installers had to be certified for overhead, sustained-load condition installations by the time the ACI 318 2011 code was published.

2.2 ACI Committee 355 Action

Overall, the ACI 355 committee agreed that creep was a contributor to the ceiling failure in Boston, but also believed that the greater problem was with the inability of the installer to provide consistent, high-quality installations in the overhead orientation. At the request of Committee 318, Committee 355 was charged with the task of leading the development of a certification program with ACI’s Certification Department. The certification program had to be developed and available to the construction industry by the issuance of the 2011 code, which at the time, was just 2.5 years away. Additionally, the committee had to have a balloted and approved adhesive anchor qualification standard, ACI 355.4 Qualification of Post-Installed Adhesive Anchors in Concrete, completed in the same 2.5-year time period. Through the great effort of the ACI 355 committee, the ACI Certification Department and a certification development committee, both efforts were completed and implemented by the publication of the ACI 318-11 Code.

3 Development of the AAI Certification Program

ACI Committee 355 Chairman Meinheit (2006-2012) also chaired the AAI certification development committee. Subject matter experts (SMEs) from Committee 355 also volunteered to participate on the development committee. It was apparent from the beginning of this process that ACI would receive strong industry support for this important program as representatives from Hilti, ITW Redhead, Powers Fasteners and Simpson Strong-Tie, already members of 355, joined the AAI certification development committee. Additionally, the Concrete Reinforcing Steel Institute (CRSI) partnered with ACI in support of this program. This development committee would move into ACI’s certification committee structure and follow established protocols for developing new certification programs.

Initial meetings of the certification committee focused on: “What does the committee want to accomplish with a certification program? What do we, as engineers, expect the installer to know? What does the certification program want to ‘measure’?” Also discussed was whether the program should be accredited or not. ACI decided to follow ANSI/ISO Guidelines in the certification development process for this new program, in the event ACI wanted to seek accreditation of the program at a future date. ACI also hired Professional Testing6, Orlando, Florida, to serve as a facilitator to fast-track program development.

3.1 Protocols for Program Development

The facilitator provided an outline of the process that would be followed by the committee. The process included the following:
1. Development of a Body of Knowledge   4. Vetting of the examinations
2. Development of written exam questions  5. Passing Score Study
3. Development of practical exam checklists  6. Pilot Programs

First the certification committee had to agree on what they expected to accomplish with the certification program. Is it just a written exam that measures the technical knowledge of the installer? Is it a performance exam that will measure installers’ ability to properly install an anchor in a horizontal and overhead installation? Or...does it include both? The committee agreed it must include both written and performance-based exams. They also agreed on the scope of this new program. The installer should have a general knowledge about adhesive anchors, understand that there are multiple systems available in the marketplace, that installation procedures and product accessories are different, and most importantly, they must read the directions provided for each product; this is covered by the objective (written) exam. The performance part of the examination was designed to measure the installers ability to Read, Comprehend, and Execute a set of installation instructions.

Development of the Body of Knowledge is a critical first step in this process. This document becomes the blueprint for the program and directs the certification committee in developing the written and practical examinations. The Body of Knowledge for the installer is usually based on reference documents that the committee members select, review, and agree upon. Interestingly, in this instance, the only “reference documents” available would be the instructions the manufacturers provide with their products. These are commonly referred to as the Manufacturers Printed Installation Instructions (MPII). The certification committee could also have obtained additional product information from each manufacturers website, if desired. However, there was no one source the committee could point to as a generic guide or workbook for the potential installer to study. The focus of the committee then became the MPII and how it would be addressed in the program. Would the MPIIs of all manufactures be used in the training and examination? Would the installer be expected to know specifics about all products? How many installation procedures would the installer need to demonstrate? It was agreed that a generic MPII would be developed by the Concrete and Masonry Anchor Manufactures Association (CAMA) and provided to ACI for use exclusively in this certification program. It was also agreed that an ACI-developed workbook would be written by the SMEs. However, the workbook development effort would take place after development of the certification program. The workbook would also use the Body of Knowledge document as its outline.

The key items in the Body of Knowledge for this program included; Preparing for Installation, Drilling Anchor Holes, Cleaning Anchor Holes, Injecting Adhesive Using Cartridge Systems, Installing Adhesive Capsule Systems, Installing Anchors. Each of these key subject areas had related subsections. The document is available at [www.concrete.org/certification/certificationprograms.aspx](http://www.concrete.org/certification/certificationprograms.aspx).
3.2 Development of Written and Performance Examinations

A 2-day meeting hosted at the Chairman’s firm, Wiss, Janney, Elstner Associates located in suburban Chicago, Illinois, was held to develop the “pool” of examination questions and the performance procedure checklist items. Participants included the ACI 355 committee members, ACI staff, and invited industry guests. The invited guests included major construction contractors that utilized and had knowledge of adhesive anchors. The facilitator assigned the members to small work groups after which each was assigned specific sections of the Body of Knowledge on which to write exam questions. The written exam was intended as a multiple-choice examination and was to contain 60 questions. The facilitator provided the SMEs with guidelines on how the questions must be written, and how to create good answer detractors. Additionally, each work group had to assign a difficulty rating to each question they created. Over the 2-day program, the teams created 120+ exam questions and a draft of the performance checklists for a vertical down and overhead installations utilizing a piston-plug system and a retaining-cap system.

Vetting the exam questions – This process includes the group review of all the examination questions to verify the following: the intent of each question; if the question is technically correct; and if there is one clear answer. It also includes a review of the proposed distractors provided in each answer set. Lastly, the review team had to agree upon the difficulty rating of the questions, as assigned by the creators, 1 being easy and 5 difficult. The goal being a blend of the difficulty levels in the eventual examination.

With this exercise complete, a written examination was assembled from the pool of questions. Committee efforts then focused on hosting a Passing Score Study for the written exam. A Passing Score Study requires actual participants from the intended target market to take the written examination. In this case, actual adhesive anchor installers with previous experience took the written exam. ACI identified major contractors across the United States and invited them to participate in this exercise. A group of ten (10) installers representing seven states and various experience levels attended the Passing Score Study hosted at the Concrete Reinforcing Steel Institute (CRSI) headquarters in suburban Chicago, in December, 2010. At this Passing Score Study, all 10 potential certified installers took the examination, rated the difficulty of each examination question, and were debriefed on the content of each examination question. Results from this independent study resulted in requiring a minimum score of 74% to pass the Adhesive Anchor Installer written examination.

Concurrently, the certification committee continued to work on the performance examination. Final requirements included installation of a 6-inch (150 mm) long, ½-inch (12.5 mm) diameter threaded anchor to a drilled hole of 4-inch (100 mm) depth. This included drilling and cleaning, injecting the adhesive, and installation of the anchor. The overhead test set-up included injecting adhesive into a 7/8-in (22 mm) diameter by approximately 8 ½-inch (216 mm) long polycarbonate tube that the installer could not see while injecting the adhesive (Fig. 4). The requirement was to fill the tube nearly void-free utilizing both the piston-plug and retaining-cap systems. The certification committee identified the pass/fail criteria for the overhead installation to be no single void larger than 3/8 x ½-inch (9.5 x 12.5 mm) in the deepest 2/3rds of the polycarbonate tube.
Fig. 4: Typical blind test set-up for overhead installation with a piston plug system being used.

3.2 Pilot Programs

A Pilot Program is a session where the written and practical exams are administered to participants that have been identified as having experience with the subject matter. No training, review, or practice session is included in a pilot program. For this pilot program the facilitator suggested 30 participants attend. ACI hosted two pilot programs in the Chicago, Illinois, area in the Winter and Spring of 2011; a total of 35 participants attended. Participants for the program were from the greater Midwest area and were identified by their employer as someone who installs anchors for their firm. Experience of the participants varied from 1 to 20+ years, with a mix of men and women and an age group from 22 to 55+ years old. Participants were asked to rate the difficulty of each examination question and agreed to be interviewed upon completion of the practical exam.

A majority of the participants believed the written exam was appropriately basic and straight forward but were not happy with the practical portion of the Pilot Exam. The primary complaint was that the participants were not given an opportunity to practice with the different equipment and systems prior to being “tested.” The participants believed they would have done much better on the performance examination and had a greater opportunity to pass the performance examination if a practice session had been offered. Many failed because they did not read the MPII directions. At the test station, each participant was given a generic MPII provided by CAMA, and was told to use those instructions for the performance examination. Some struggled to follow instructions. Issues like putting the wrong accessories on extension tubing, not expressing out an initial amount of material when starting with a new cartridge, or overfilling of the drill hole or plastic tubes were observed (see Figure 5). Recall that all had experience with anchor installation, yet 80% of them failed the practical exam and 30% of them failed the written exam. The average score on the written exam was 81%.
Fig 5: Examples of failures with overhead methods. Left to right, Piston Plug upside down and left in tube, Retaining Cap with 1+ inch void in center of tube, voids at top and middle of tube\(^7\).

A grading guide, or rubric, for evaluation of the filled overhead tubes was created and used to evaluate the tube samples from the pilot programs. This rubric was subsequently distributed to all ACI test administrators offering the AAI program.

With only minor changes to the performance examination set-up and execution, the program was officially declared ready to go, and roll-out was set to begin in the summer of 2011, several months ahead of the publishing of the ACI 318-11 Code.

3.2 Development of AAI Workbook

In late 2011, ACI staff and SMEs from the AAI committee again began work this time on an Adhesive Anchor Installer workbook. The Body of Knowledge document was used as the outline of the workbook, the committee also utilized the MPII’s from the manufacturers, other general technical information from the industry, and two Concrete International magazine articles about Adhesive Anchor Installations. The workbook (see cover page in Figure 6) was completed in August 2012. The 95-page workbook contains 9 chapters, review questions after each chapter, performance checklists, a practice exam, and a 22-minute training video on proper procedures for both downward and overhead adhesive anchor installations.

Fig. 6: ACI/CRSI Adhesive Anchor Installer workbook\(^8\).
With the delivery of the AAI workbook, passing rates for the written exam improved and the average score on the written exam also improved to 87%. Currently, the workbook and exams are available in English, Spanish, Mandarin, and French.

4 Program Rollout via ACI Sponsoring Group Network

ACI’s Sponsoring Group network provides excellent coverage throughout North America. In most instances Sponsoring Groups are connected with a local ACI Chapter and the local chapter members organize, staff, and execute ACI Certification programs for their local concrete community. With ACI offering over 20 concrete-related certification programs, the Sponsoring Group network is critical in getting these programs to the end users. ACI Certification programs cover a wide area of services including Concrete Field and Laboratory Technicians, Inspectors, Supervisors and Craftsmen. Each Sponsoring Group offers select programs that are needed in their market and schedule such programs throughout the year. Generally, programs offered by the Sponsoring Groups are typically 2-days long and include a review and hands-on practice session on Day 1 followed by the written and practical examination on Day 2. As learned from the AAI Pilot Programs, ACI requires all Sponsoring Groups offering the AAI program provide a review and hands-on practice session prior to administering the examinations. With 120 Sponsoring Groups worldwide, and 97 Sponsoring Groups operating in North America, the ACI Certification Department has prepared 40+ to offer the AAI program.

![Fig. 7: Examiner reviews filled tubes with an Installer at practice session](image)

A Sponsoring Group must show an interest in offering the AAI certification program. Members of the group are required to go through an ACI orientation program including written and performance exams. The program policy for the AAI program requires the Examiners to be AAI-certified and to achieve a passing score of 90% or greater on the written exam. ACI Certification Department staff provide on-site training to the hosting Sponsoring Group and lead an initial AAI training and examination session explaining all the critical issues associated with the program. Included in the orientation program is instruction on how to conduct a review session, test station set-up, product lines and accessories, and inspection of the tubes after curing (see Figure 7). These orientation programs usually have between 5 and 8 representatives from the local sponsoring group that volunteer to participate in bringing the program to their market area. ACI also invites local technical-
service representatives from Hilti, ITW Redhead, Powers, and Simpson, to participate, and all are very supportive of this program. An AAI program guide was developed for a potential Sponsoring Group so they can plan for everything involved with set-up and execution of this certification program prior to deciding to add it to the list of certifications they offer.

*Mandated certification by code - 318-11, Building Code Requirements for Structural Concrete* was published in July 2011. In Appendix D, Section 9.2.2 (and Chapter 17 Section 17.8.2.2 of ACI 318-14) states;

*D.9.2.2 (17.8.2.2) — Installation of adhesive anchors horizontally or upwardly inclined to support sustained tension loads shall be performed by personnel certified by an applicable certification program. Certification shall include written and performance tests in accordance with the ACI/CRSI Adhesive Anchor Installer Certification program, or equivalent.*

From July 2011 to March 2015, ACI staff completed 40 Sponsoring Group/Chapter orientation programs throughout North America (see Figure 8) as well as South America and Taiwan. During this time, numerous articles, seminars, presentations were also scheduled and executed. Additionally, in January of 2014, the ICC Evaluation Services updated all their Engineering Service Reports for adhesive anchors to include language to reflect design provisions of ACI 318-11. Specifically, the ICC calls out the ACI/CRSI Adhesive Anchor Installer program.

![Fig. 8: ACI Sponsoring Groups currently offering the AAI certification program.](image)

### 5 Need for AAI Inspector Program

As the AAI certification program began to generate interest, it became apparent to many of the program Examiners as well as ACI 318 committee members, that a needed component was a program for Special Inspectors. A number of Inspectors were going through the AAI program in an effort to gain specific knowledge about adhesive anchors and to know what the installers were supposed to be doing as certified installers.

During Spring 2014, the ACI C680 *Adhesive Anchor Installer* committee discussed this topic and a task group was formed to investigate the potential need and content of an inspector program. By Fall 2014, C680 submitted a request to develop an inspector program. A new certification program development committee was created, resulting in a new AAI Inspector certification. All the subject
matter experts came from the C680 committee, and in less than 30 months, the Inspector program was completed and available to the industry. Program development followed the same protocols as the AAI program with the exception that no workbook was required to be developed. All the reference documents were previously published and used on a regular basis by inspectors, making it much easier for the SMEs to develop the Body of Knowledge for the Inspector program. Four (4) Pilot Programs were organized for the Inspector program with over 80 inspectors participating. Sessions were held in New York City, Philadelphia, Chicago, and the Oakland/San Francisco Bay Area. The pilot programs were successful with very positive feedback from the inspectors. Inspectors particularly liked the required hands-on demonstration part of the program. Interest in this program is very high, and growing rapidly including government agencies seeking to specify it.

6 Lessons Learned

Assembling this program and conducting the orientation sessions served as an eye opener for all involved regarding potential difficulties with overhead installations. This program experienced some difficulties during development and the members worked hard to correct the procedures to make the program viable. Particularly the overhead installation procedures had to be modified from their original concepts.

7 Conclusion

ACI and its members actively responded to an important industry need. ACI committees reviewed the situation and acted on multiple levels to prevent recurrence of the errors leading to the Boston tragedy. Changing codes and standards, developing and hosting educational seminars, and development of a thorough certification program that measures the physical skill set as well as the technical knowledge of the installer is taking the industry in the right direction. This certification program ensures that the installer is fully aware of their responsibilities when using adhesive anchors.

Unfortunately, as important as it is, this program has been met with some resistance in North America. Even though it is required in the ACI 318 Code, many contractors do not have certified installers in their labor pool, viewing this required certification only as an “added cost.” As yet, few are policing this important issue. If contract documents don’t include requirements, or if included requirements aren’t enforced, anchors will continue to be installed by unqualified personnel. How shortsighted…do we have to wait for another failure, another death to enforce this?

With availability of the new Adhesive Anchor Installation Inspector certification program, ACI anticipates an increase in demand for the AAI program generated by increased awareness and enforcement. Further assisting the increase will be the adoption of updated codes requiring certified installers and inspectors.

8 Acknowledgements

The authors would like to thank all the members of the ACI Committee C 601-A Adhesive Anchor Installer, the members of ACI Committee 355 Anchorage to Concrete and ACI staff for their commitment and diligence in developing these important new standards and programs. They would also like to acknowledge the support received from the adhesive manufacturers, specifically: Hilti,
Michael Morrison, Donald F. Meinheit, and Neal S. Anderson

ITW Redhead, Powers Fasteners and Simpson Strong-Tie as well as the Concrete Reinforcing Steel Institute.

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ANCHORING INTO CONCRETE – THE AUSTRALIAN EXPERIENCE

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ABSTRACT

Prior to 2012, the Australian anchor industry was fragmented; different terminology, notation, evaluation procedures for products and design guidelines existed among suppliers, making selection and design of fasteners confusing to engineers and potentially dangerous. The Australian Engineered Fasteners and Anchors Council (AEFAC) a collaborative initiative between industry and Swinburne University of Technology was founded in 2012 to bring uniformity into product prequalification, design and installation and to align Australian practice with international best practice. AEFAC’s work has revolutionised Australia’s anchor industry with the introduction of a new Technical Specification TS 101 which is the first design provisions for post-installed and cast-in fasteners in Australia. Also a new Installer Certification Program has been developed and introduced to equip installers with knowledge and skills on best practice installation. Ongoing research is currently being conducted in a number of areas related to the Australian construction practice including anchoring into early-age concrete, close to edge and in high strength concrete. This paper discusses the Australian journey into uplifting the quality and safety standard of anchoring into concrete, current research being undertaken to address knowledge gap in the industry and the safety framework that has been introduced for reliable performance of fasteners to concrete in safety-critical applications.

1 Background

The absence of suitable guidelines for anchors in safety-critical applications has contributed to catastrophic failures worldwide. Among others are failures of precast panels during erection in Victoria, Australia (Figure 1 and Figure 2), canopy-fence failure on 17th Street Bridge in Atlanta, U.S.A. (Figure 3), awning collapse in Queensland, Australia (Figure 4) and partial collapse of the Sasago tunnel ceiling in Japan (Figure 5).

The cases mentioned above demonstrate the safety-critical nature of anchors and the importance of learning from mistakes that have resulted in failures. A number of common themes are present:

- **Product prequalification:** A robust product approval process ensures the product is fit for purpose. The cast-in ferrule that failed (Figure 1) not only lacked traceability, its failure during installation also demonstrated excessive variability in product performance that was not controlled to an acceptable level.
Product substitution and change management: A proper change management procedure involving written consent by the responsible engineer must be followed for any deviation from the specification. The 17th Street Bridge in Atlanta (Figure 3) had threaded rod substituted for only partially threaded rod, reducing mechanical interlock of the anchor rod with the chemical product that compromised performance and contributed to the collapse.

Quality of installation: All fasteners to concrete exhibit some degree of sensitivity to installation such that their performance is limited to a certain degree by the quality of their installation. The installer needs to understand the dangers of deviating from the installation instructions. Poor installation technique for chemical anchor installation was identified in the 17th Street Bridge in Atlanta resulting in chemical compound that had not cured and the presence of large air voids. The collapse of the awning (Figure 4) highlighted the danger of reducing drilling depth due to hitting reinforcement since the fastener cannot achieve its rated performance at the reduced depth.

Accurate and detailed specification: A specification with insufficient information may lead to the same type of product being procured that exhibits very different performance. The limited information provided in the specification for chemical anchors for use in the 17th Street Bridge in Atlanta resulted in the potential to procure 15 different products. Further, the need to avoid creep-sensitive chemical product was not identified. A proper understanding of project-specific requirements is imperative for fastener specification by the responsible engineer.

Inspection and maintenance program: In the event that defects do occur, an inspection and monitoring program provides an early detection mechanism and the opportunity to undertake remedial work before defects escalate to a collapse situation. The corrosion of fasteners securing the awning that collapsed in Queensland had been in service for more than three decades but could not be easily inspected. Findings reported from the preliminary investigation into the Sasago tunnel collapse (Figure 5) revealed an inspection and monitoring program had not been enacted during the 12 years preceding the collapse and that during the ensuing investigation 1028 defects were found in the fasteners securing the concrete panels remaining in the tunnel that experienced the collapse.
(a) Canopy fence collapsed onto roadway.  
(b) Threaded and smooth anchor rods.  

Figure 3: Canopy fence failure on Interstate Highway 75/85 in Atlanta (source: 3).

(a) Structure that swung down.  
(b) End of awning with anchors showing.  

Figure 4: Awning collapse in Queensland (source: 4).

Figure 5: Partial collapse of concrete panels in Sasago Tunnel (source: 5).
Recognising that the shortcomings mentioned above were prevalent in Australia, the Australian Engineered Fasteners and Anchors Council (AEFAC) initiated in 2012 by Swinburne University and formed with the support of the 6 other Founding Board Members aims to mitigate such failures by being proactive in enhancing the specifications, design requirements and installation of anchors. Internationally, standards and training have been implemented only in response to catastrophic or tragic events; AEFAC’s goal is to be ahead of such consequences for Australia. Figure 6 summarises the activities taken by AEFAC for quality assurance in products prequalification, design and installation.

![Diagram of AEFAC's activities](image)

**Figure 6: Work undertaken by AEFAC to achieve quality assurance**

## 2 Design Guidelines for Australia

The anchor industry in Australia has been self-regulated, relying on suppliers of products for information and performance data. However, this is often confusing for engineers due to there being a lack of consistency in testing and evaluation of products, as well as differences in terminology and design methodology. Unlike structural bolts or welds, the design of anchor connections and evaluation of anchor products in Australia were not unified until SA TS 101 was published in 2015. SA TS 101:2015 is an initiative of AEFAC providing the first design provisions for post-installed and cast-in fasteners in Australia that has been referenced in the 2016 National Construction Code as part of national regulations.

The methodology underpinning the TS 101 is the Concrete Capacity (CC) Method that has been adopted in Europe and the United States for the design of cast-in and post-installed anchors.
2.1 Scope
The SA TS 101:2015 provides the minimum requirements for the design of fastenings used to transmit loads to concrete for safety-critical applications. The term ‘safety-critical’ denotes a risk to the welfare of people or considerable economic loss in the event of failure. In the case of fasteners, the responsible engineer will determine whether a certain application is safety-critical.

TS 101 covers the design of post-installed fasteners (chemical and mechanical anchors) and cast-in anchor channel for safety-critical applications. The document provides guidelines for the determination of forces acting on fasteners, taking into account eccentricity of loading on fasteners and prying forces. The most loaded fastener is identified and designed for. The SA TS 101 provides guidelines to check for various modes of failure under tension (refer Figure 7) and shear (refer Figure 8) loading respectively. The critical mode of failure producing the lowest design strength will be the one governing the design tensile and shear capacity of the fastener. If a fastener is subjected to an oblique load, the fastener should also be checked for a combined tension and shear actions. Other design considerations include influence of concrete edges, influence of a lever arm for shear loading, influence of fixture plate and load resisted by supplementary reinforcement (if present) to be designed in accordance with the Australian concrete code, AS 3600:2009.

Figure 7: Failure modes in tension
2.2 Exclusions

The SA TS 101 does not cover the design for the following items:

- exposure to fire, durability and seismic actions
- design of fixtures
- fasteners for lifting, transport and erection (brace inserts, lifting inserts, etc.). A separate Australian Standard (AS 3850:2015) is dedicated to precast concrete which covers the prequalification and design of such fasteners.
- headed fasteners
- ferrules
- reinforcement for development length considerations
- headed reinforcement
- anchorage for prestressing strands

TS 101 covers the design of shallow embedments with depth not exceeding 20d_{nom} for chemical fasteners (where d_{nom} = outside diameter of fastener). For embedments greater than 20d_{nom} design of development length to develop yield strength of the bar can be done according to AS 3600:2009 or Eurocode 2\textsuperscript{13}. If designing in accordance with Eurocode 2, the chemical fastener is required to be prequalified to TR 023\textsuperscript{14}.
2.3 Product prequalification

Product prequalification is one of the major components in the safety framework for anchors where a robust product approval process ensures the product is fit for intended purpose. The national regulations in Australia are performance based and thus do not allow for approvals such as an European Technical Assessment (ETA) or ICC-ESR to be made mandatory as no one body can be deemed the approving authority for a building product. Product prequalification requirements is made mandatory in TS 101 without referring to specific approvals but specifying the criteria to be met. Post-installed fasteners are required to be tested and assessed in accordance with the requirements of Appendix B in the document which refers to the testing procedures outlined in European Technical Assessment Guideline, ETAG 001 Part 1 to Part 5. Meanwhile, cast-in anchor channels are required to be tested and assessed in accordance with the requirements of the European Assessment Document (EAD) “Anchor channels”. It is highlighted in TS 101 that anchor products that have been awarded ETAs automatically satisfy the requirements of Appendix B for product prequalification.

2.4 Specification

The SA TS 101 recognises the importance of complete and proper specification. Appendix E in the document refers to the AEFAC Technical Note “Guideline for the specification of fastenings in engineering general notes”. The AEFAC Engineering General Notes provides guidelines for specifications of fasteners in engineering drawings to prevent ambiguity and outlines the change management for product substitution. Current standard notes on engineering contract drawings frequently allow the use of ‘equivalent product’ in lieu of the fastener specified by the design engineer. It is recommended that the responsible engineer must be consulted for the approval of an alternate product or for the approval of a revised specification in the event that the fastener cannot be procured or installed as per the original specification. Alternate products must have the appropriate prequalification and design. Figure 9 provides an example of minimum information to be specified in engineering drawings for chemical fasteners.

<table>
<thead>
<tr>
<th>Chemical</th>
<th>Manufacturer’s name</th>
<th>Product name</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor rod</td>
<td>Type</td>
<td>(E.g. Threaded rod)</td>
</tr>
<tr>
<td></td>
<td>Diameter</td>
<td>(E.g. M12)</td>
</tr>
<tr>
<td></td>
<td>Length (mm)</td>
<td>(E.g. 200mm)</td>
</tr>
<tr>
<td></td>
<td>Finish/Coating</td>
<td>(E.g. Galvanised)</td>
</tr>
<tr>
<td></td>
<td>Strength Grade</td>
<td>(E.g. Class 8.8)</td>
</tr>
<tr>
<td></td>
<td>Depth of embedment (mm)</td>
<td>(E.g. 110mm)</td>
</tr>
<tr>
<td>Drill hole</td>
<td>Diameter (mm)</td>
<td>(E.g. 14mm)</td>
</tr>
<tr>
<td></td>
<td>Depth (mm)</td>
<td>(E.g. 110mm)</td>
</tr>
<tr>
<td></td>
<td>Drill type</td>
<td>(E.g. Carbide-tipped)</td>
</tr>
<tr>
<td>Max tightening torque (Nm)</td>
<td>If applicable</td>
<td>(E.g. 100 Nm)</td>
</tr>
</tbody>
</table>

Figure 9: Sample specification for chemical or bonded anchors
The AEFAC Engineering General Notes\textsuperscript{16} covers specification requirements for other types of fasteners such as torque controlled expansion fastener – thick walled and stud type, deformation controlled expansion fastener, concrete screw and cast-in anchor channel.

3 Installer Certification Program

The AEFAC Installer Certification Program has been developed to promote best practice for installation to ensure safety and is recommended by SA TS 101. The program is based on the American Concrete Institute ACI-CRSI Adhesive Anchor Installer Program\textsuperscript{17}, but it has been extended to include mechanical anchors and adapted for Australian practice.

The Installer Certification Program is a full day program consisting of a half day of face-to-face training, followed by a written examination and a practical examination as shown in Figure 10. During the training, installers are introduced to the different types of post-installed anchors and their application matrix, mechanics of anchors and failure modes. Photos from practical exam are shown in Figure 11 for vertical down installation and Figure 12 for overhead injection.

Certification is awarded to installers upon passing both the written and practical exams. Recertification is required after a three-year period and after that every five years to ensure that certified installers have the current skills and knowledge. AEFAC Certified Installers can be identified by having a current Certified Installer card (sample shown in Figure 13) and names listed on the AEFAC website under list of certified installers.

![Figure 10: AEFAC Installer Certification Program](image-url)

Figure 10: AEFAC Installer Certification Program
Field testing or site testing is often specified by design engineers to validate correct installation (proof tests), or to identify the characteristic strength of a fastener in a given substrate (ultimate tests). The AEFAC Technical Notes on site testing, Volume 1 to 4\textsuperscript{18,19,20,21} provide best practice recommendations for site testing of post-installed anchors and is in line with international practice recommended by BS 8539\textsuperscript{22} and CFA\textsuperscript{23}.

The following load regime is proposed for proof test in Australia which is slightly different from the recommendations in BS 8539\textsuperscript{22} and CFA\textsuperscript{23}:

- For anchors with ultimate limit state capacities, proof test load ($N_p$) should not exceed the lesser of the ultimate limit state design capacity, $\varnothing N_{rk}$ of the anchor and 0.7x yield capacity of the anchor to prevent damage to the anchor. The capacity reduction factor $\varnothing$ is 0.67 for concrete and $N_{rk}$ is the characteristic resistance of anchor.
- For anchors with allowable working load capacities, proof test load ($N_p$) should not exceed the lesser of 1.5 x allowable working load of the anchor and 0.7x yield capacity of the anchor to prevent damage to the anchor.
The proof load shall be maintained for a minimum of 30 seconds. The load should not drop more than 10% in that duration.

Proof load is taken as a force equivalent to a fraction of the design capacity of the anchor to ensure that the anchor remains serviceable after the proof test. Depending on the type of anchor, at the proof test load, there could be visible displacement. For some anchors, this could constitute a failure whereas for others (such as torque controlled anchor), this may not be considered to be a failure but would exceed the allowable serviceability limit. For applications where serviceability function is critical for the anchor, displacement measurements should be taken and the displacement acceptance criteria should be assessed by the responsible design engineer.

5 Current Industry Trends in Australia

5.1 Performance of anchors in early age concrete
There has been increasing use of prefabrication in the Australian construction industry which contributed to a very strong drive in the industry for early lifting of precast elements to reduce construction time cycles. Precast elements are predominantly lifted within the first 24 hours and the lifting design relies upon the properties of concrete at this early age. AEFAC was awarded a Linkage Grant by the Australian Research Council (ARC) to investigate performance of anchors in early age concrete. Research is currently taking place at Swinburne University of Technology. The research done from this project aims to improve safety for early lifting in precast industry and provide economic efficiency to precast concrete mix design if strength of anchors in early-age concrete can be determined easily and reliably.

Research is also conducted at Edith Cowan University on performance of screw anchors in early age concrete. Time-dependent behaviour of concrete such as early age creep and cracking in concrete is investigated by University of New South Wales and University of Sydney. Research into early age precast edge lifting was conducted by Curtin University previously. Local manufacturers and suppliers are also continuing to design and develop products for specific applications in Australia such as lifting and support of tilt up panels.

5.2 Seismic requirements of anchors in Australian conditions
The scope of the design provisions for fastenings to concrete in Australia that are provided in Standards Australia TS 101:2015 do not cover design verification for seismic loading. Current research is being conducted by AEFAC to investigate the European and US seismic prequalification and design requirements for adaptation to suit Australian requirements. Work is currently in progress to identify suitable Australian seismic hazard level categories corresponding to fastener seismic performance categories C1 and C2. (ETAG 001).

5.3 100 years design life for anchors
The Australian Bridge Design Code AS 5100 requires that road structures, particularly new structures, have a 100-year design life. Anchor products prequalified in accordance with ETAG 001 are generally intended to have a 50-year design life. Anchor suppliers may be able to provide additional data beyond the prequalification information based on proprietary testing or experience. In
general, for bonded anchors one of the major concerns is their creep performance under sustained loading over time. Current provisions in TS101 and associated ETAG testing are limited to 50 years. In order to estimate the maximum creep displacement over a longer period, additional testing may be required. One such example is the Legacy Way tunnel in Brisbane\textsuperscript{30}, where specific testing was conducted for chemical anchors comprising a 20mm rebar installed to a depth of 300mm into steel fibre-reinforced concrete segments. The specimens were placed in an environmental chamber for over a period of 3 months with sustained loading of 125kN at 20°C +/- 5°C. The deflection was extrapolated to 100 years in accordance with ETAG 001\textsuperscript{15} Part 5 Section 6.1.1.2 to check for compliance. Studies by Eligehausen et al.\textsuperscript{31} on the behavior of chemical fasteners under sustained load found that the projection of displacements from a relatively short time to the intended service lifetime using current formulation in ETAG 001 is conservative. It is proposed that for a 100-year design life, the projected displacement should not exceed the mean value of displacements at loss of adhesion in the corresponding reference tests at normal ambient temperature or maximum long-term temperature. In addition to satisfying this maximum limit, the fastening system should also satisfy durability requirements which would depend on the environmental conditions for the nominated application. Ongoing work is conducted to provide guidance for long term performance of anchors for Australian conditions.

6 Conclusions

AEFAC was founded in 2012 as an industry initiative to lift quality and safety standards to prevent failures in the Australian anchor industry. All AEFAC members are unified by a common goal to protect life safety in an industry essential to the Australian economy and community. The pace that AEFAC has progressed since its inception is unprecedented in this industry. Not only has a certification scheme been developed for installers which will uplift in the standard of installation practices leading to an overall safer and more reliable structures, but the technical specification TS 101 for design engineers introduces certainty in product compliance and provides leading international methodology for design. This was published in 2015. Both of these initiatives are a first in Australia.

Research is currently in progress to identify future modifications to TS 101 to include design of fastenings to concrete for seismic actions within its scope and to provide guidance on design of deep embedment based on development length calculations in Australian concrete standard AS 3600 and Eurocode 2. Work has also commenced in providing guidance for the performance of anchors beyond 50 years of service life as bridges in Australia are designed to 100 years of service life. Ongoing research on performance of anchors in early-age concrete is being conducted as part of the industry drive to improve safety of lifting precast concrete elements to reduce construction time cycles.

7 Acknowledgement

The authors would like to acknowledge the technical input and ongoing financial support from the AEFAC Founding Members: Ancon Building Products, Hilti (Aust.), Hobson Engineering Co., ramsetreid, Stanley Black and Decker (Powers) and Wörth (Aust.), and Supporting Members: Allthread Industries, Simpson Strong-Tie Australia and Iccons Pty. Ltd.
References:


12. Standards Australia. AS 3600: Concrete Structures. SAI Global, Sydney, 2009

13. British Standard, Eurocode 2: Design of Concrete Structures (EN 1992-1-1), British Standards Institution, 2004


17. American Concrete Institute, “ACI-CRSI Certification Program for Adhesive Anchor Installer”, Publication CP-80 (12), www.concrete.org


29. Standards Australia. AS 5100: Bridge Design. SAI Global, Sydney, 2004


PRODUCT EVALUATION REPORTS FOR ANCHORS IN THE U.S.

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ABSTRACT

The basis for evaluation reports for anchors in the US is the International Building Code (IBC)\(^1\) (the dominant model code in the U.S.), Section 104.11 - Alternate materials, design and methods of construction and equipment. This long-standing code section provides the mechanism for acceptance of products, methods and materials not specifically referenced in the code by the authority having jurisdiction (usually, the local building official). The section refers to research reports as follows: “Supporting data, where necessary to assist in the approval of materials or assemblies not specifically provided for in this code, shall consist of valid research report from approved sources.” (Section 104.11.1). Approved source is further defined as “An independent person, firm or corporation, approved by the building official, who [sic] is competent and experienced in the application of engineering principles to materials, methods or systems analyses.” Thus, the code makes no specific reference to evaluation reports, but rather to research reports from approved sources. It is understood throughout the building community, however, that, e.g., Evaluation Service Reports from ICC-ES or Uniform Evaluation Reports from IAPMO UES constitute a ‘research report’ from approved sources. This is in fact the basis for all product evaluation reports issued by agencies such as IAPMO, ICC-ES, Intertek, UES, UL, and others.

The manner in which these reports are generated for anchors in concrete and masonry, the criteria used for their development, and the way in which they should be read and understood, is the subject of this paper. In particular, emphasis will be placed on the necessary components for a proper evaluation of an anchor system and the critical components of the report that the specifying engineer, inspector, and building official should be aware of to ensure the intended level of structural safety in the built environment.

1 Introduction

Post-installed and cast-in-place (CIP) anchors in concrete and masonry are widely used in the construction industry and are an important part of modern construction. However, proprietary anchorage systems are becoming increasingly complex and their use in design and their installation requires attention to detailed and product-specific documentation. All information required for the design and the proper installation of a proprietary anchorage system is presumed to be included in manufacturer-provided documentation. The specific instructions for installation of the anchor are usually provided in the product packaging and are referred to in the code as the Manufacturers Printed Installation Instructions, or MPII. Nevertheless, evidence of code compliance, otherwise
known as conformity assessment, must be provided in a separate document that instructs the building official or other Authority Having Jurisdiction (AHJ) regarding the specific conditions under which a product may be considered as code-compliant.

In the International Building Code (IBC)\(^1\), Section 104.11 *Alternative materials, design and methods of construction and equipment* has historically been the basis for the acceptance by the AHJ of alternative building materials, design methods and construction methods that are not specifically addressed in the code. Section 104.11 reads as follows:

> The provisions of this code are not intended to prevent the installation of any material or to prohibit any design or method of construction not specifically prescribed by this code, provided that any such alternative has been approved. An alternative material, design or method of construction shall be approved where the building official finds that the proposed design is satisfactory and complies with the intent of the provisions of this code, and that the material, method or work offered is, for the purpose intended, not less than the equivalent of that prescribed in this code in quality, strength, effectiveness, fire resistance, durability and safety. Where the alternative material, design or method of construction is not approved, the building official shall respond in writing, stating the reasons why the alternative material was not approved.

(Italicized words in code citations are defined terms in the code.)

It should be noted that the building official approves alternates and is not compelled to do so, but must, if refusing to approve an alternate, state in writing the reason for doing so.

Section 104.11.1 *Research Reports*, defines a path for acceptance based on a document prepared by an approved source as follows:

> Supporting data, where necessary to assist in the approval of materials or assemblies not specifically provided for in this code, shall consist of valid research reports from approved sources.

The term “approved” is defined in Section 201 as simply “Acceptable to the building official.”

Thus, use of an alternative to a material or method in the code is dependent on approval by the building official and such approval may be based on valid research reports, which are research reports from approved sources, which in turn are sources acceptable to the building official.

This paper will explain in detail how such research reports, commonly known as *evaluation reports*, are generated, the criteria on which they are based, and the manner in which typical reports are structured and read.
2 Evaluation reports and assessment criteria

Currently, only two institutions issuing research reports for anchors in concrete and masonry in the US are accredited under ISO 17065 – *Conformity assessment – Requirements for bodies certifying products, processes and services*. These are: the IAPMO Uniform Evaluation Service and the ICC Evaluation Service, LLC.

**Table 1 – Basis for assessment of anchors**

<table>
<thead>
<tr>
<th>Standards for the Assessment and Design of Anchors in Concrete and Masonry</th>
<th>Acceptance Criteria for the Testing and Assessment of Anchors in Concrete and Masonry</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 318: Building Code Requirements for Structural Concrete</td>
<td>AC01: Mechanical Anchors in Masonry Elements</td>
</tr>
<tr>
<td>ACI 355.2: Qualification of Post-Installed Mechanical Anchors in Concrete</td>
<td>AC58: Adhesive Anchors in Masonry Elements</td>
</tr>
<tr>
<td>ACI 355.4: Qualification of Post-Installed Adhesive Anchors in Concrete</td>
<td>AC60: Anchors in Unreinforced Masonry Elements</td>
</tr>
<tr>
<td>ASTM E488: Standard Test Methods for Strength of Anchors in Concrete Elements</td>
<td>AC106: Predrilled Fasteners (Screw Anchors) in Masonry</td>
</tr>
<tr>
<td></td>
<td>AC308: Post-installed Adhesive Anchors in Concrete Elements</td>
</tr>
<tr>
<td></td>
<td>AC398: Cast-in-place Cold-formed Steel Connectors in Concrete for Light-frame Construction</td>
</tr>
<tr>
<td></td>
<td>AC399: Cast-in-place Proprietary Bolts in Concrete for Light-frame Construction</td>
</tr>
<tr>
<td></td>
<td>AC446: Headed Cast-in Specialty Inserts in Concrete</td>
</tr>
</tbody>
</table>

ISO 17065 specifically requires that the conformity assessment body maintain, and make available upon request, “…information about (or reference to) the certification scheme(s), including evaluation procedures, rules and procedures for granting, for maintaining, for extending or reducing the scope
of, for suspending, for withdrawing or for refusing certification.” Furthermore, under Sec. 7.7.1 (d) the certification body is required to disclose of the scope of certification, which is defined as identification of “…the standard(s) and other normative document(s), including their date of publication, to which it is judged that the product(s), process(es) or service(s) comply.” This requirement is the basis for the development of criteria which are published by the conformity assessment bodies using publicly accessible fora.

The assessment of anchors for use in concrete and masonry with regards to qualification and design is based upon standards produced by the American Concrete Institute (ACI), the National Concrete Masonry Association (NCMA), the American Society for Testing and Materials (ASTM) and other organizations. In addition, acceptance criteria for the assessment of anchors in concrete and masonry are maintained by, e.g., the ICC Evaluation Service. See Table 1.

3 General organization and components of evaluation reports

The format of evaluation reports is not regulated but typically follows a fixed structure that has evolved over the past decade. Most anchor reports contain the following organizational elements (not necessarily in this order):

Report holder - provides information about the report holder, which may be the distributor or the manufacturer of the anchor product.

Evaluation subject and scope - lists the proprietary product by name (as it appears on the product packaging) and the building codes addressed by the report.

Use of product – provides information regarding, e.g., applicable Seismic Design Categories, concrete strength range, characterization of the concrete as uncracked or cracked, and specific applicable code sections.

Description - lists sizes, types, and all components and accessories for the proprietary product or system as well as a facsimile of the MPII (Manufacturers Printed Installation Instructions).

Design, installation, and inspection requirements - contains essential information for the designer, the installer, and the special inspector. It relates each design strength parameter to the appropriate design code section. Where specific code provisions for design of the subject anchor do not exist, this section may include design provisions and equations that either supplement or replace those in the code. The text of this section refers to the associated tables and figures of the report. Required design checks are described in detail and additional explanation is given where necessary for the safe design of the anchor, including restrictions on anchor spacing, edge distances, member thickness, etc. Specific information is also provided for correct installation and inspection of the anchor, e.g., cases requiring installer certification.

Conditions of use – provides a summary of the special conditions and restrictions for the use of the proprietary anchor or system, and thus provides guidance for the correct specification or, where applicable, substitution of anchors.
Evidence submitted – an often-overlooked part of the report; provides the basis for the evaluation of the product or system, usually in terms of the applicable acceptance criteria (“data submitted in accordance with…”). Since acceptance criteria as maintained by the respective evaluation agencies can be modified on short notice, it is important to note the revision date of the criteria applied.

Identification – primarily for the inspector; provides information used to facilitate identification of the product on site.

Tables – for anchor parameters and anchor design values. The design information given in each table is derived from application of the assessment criteria to the test results.

Figures – typically include renderings of the anchor and accessories as well as details indicating the installed condition with callouts for effective embedment depth, anchor cross section (e.g., as critical for shear) and other critical dimensional aspects.

Installation instructions – are intended to be a one-to-one graphical representation of the MPII as provided in the product packaging and as used by the test laboratory for installation of the anchor in the assessment tests. This is vital to ensure that the installed product on site correlates to the product as assessed, and that the design values employed by the engineer of record are in line with the intended safety margin included in the assessment criteria and design provisions.

Design example – may be provided at the option of the report holder where the application of the design provisions, equations and tabulated parameters included in the report is not self-explanatory.

It should be noted that, in many cases, the technical content of the evaluation report is generated by the report holder (e.g., manufacturer). The evaluation report must conform to the contents of an assessment report generated by the primary evaluation laboratory. The assessment report is held as confidential information pertaining to the proprietary product and is retained by the evaluation agency. The evaluation agency reviews the assessment report and ensures that the text of the evaluation report is in compliance with the criteria listed under Evidence Submitted.

Where anchors are sold under more than one label (so-called second label products), evaluation reports are typically required for each label to obtain building department approval for use in construction. These are often referred to as “follower reports” and typically vary from the “master report” only in the description of the product packaging.

4 Stakeholders

There are essentially three classes of stakeholders in the process of requesting, generating, and maintaining evaluation reports for anchors. These are discussed briefly here.

AHJs - As previously noted, evaluation reports are typically required by building officials and other regulatory entities (more generically referred to as Authorities Having Jurisdiction or AHJs) as evidence of product conformity with code requirements. As such, manufacturers of anchors that are to be used in safety-related applications are usually required to obtain evaluation reports for those products to be successful in the marketplace. The reports are also, as noted, relied upon by the special inspector retained by the owner or owner’s representative to conduct anchor inspections in accordance with the Statement of Special Inspections.
EORs - Although primarily intended to serve as evidence for code compliance by jurisdictional authorities, in many respects *evaluation reports* are written for design engineers. They are in fact generally viewed as independent sources of reliable design information that supersede all other technical information that may be associated with the anchor, even that issued by the product manufacturer. For the design Engineer of Record (EOR), this distinction is critical. The engineer that stamps and signs project drawings and/or specifications utilizing propriety anchors bears ultimate responsibility for the safety of the constructed work, and as such, for the safety of the specified anchors. Since design engineers rarely if ever have the resources to investigate performance claims made by manufacturers of anchors, they are forced to rely on the accuracy and integrity of the applicable *evaluation reports*.

Manufacturers – The anchor manufacturing community has long recognized the value of independently audited design information for anchor products as a safeguard against unfounded and inaccurate product claims. The first acceptance criteria issued by ICBO in the 1970s was for post-installed anchors in concrete. The maintenance of a so-called “level playing field” both ensures fair competition within the manufacturing community and promotes a high standard of care for the development and assessment of safe and reliable anchor systems.

5 How to read an evaluation report

The contents of the evaluation report have varying degrees of relevance corresponding to the interests of the stakeholder. The instructions given here are intended to apply to designers but also have relevance for AHJs and inspectors.

1. Start with the product description and verify that the product addressed by the report is indeed the anchor in question. Many anchors come in a variety of flavors that are distinguishable only by the complete product name. Refer to the assessment criteria listed under *Evidence Submitted* (or analogous section) to ensure that the criteria are up to date and address the applicable code(s) and code versions. Under *Conditions of Use* confirm that the boundary conditions (e.g., type and direction of loads acting on the anchor system, Seismic Design Category, environmental conditions, concrete or masonry type) are covered by the report.

2. Review the specific provisions under *Design, Installation and Inspection Requirements* carefully to ensure that the specific application of the anchor is addressed. Where questions arise regarding specific provisions, these may be addressed to the issuing evaluation agency or directly to the manufacturer. Note that in many cases, anchor design software provided by the manufacturers automates the application of the design information for specific design conditions. The agreement of the *evaluation report* data with the output from such software should be at least spot-checked, perhaps by running the example problem provided in the report. Special requirements on the suitable additional equipment must be provided for the installation.

3. Compare the project documentation (general notes, drawings, specifications, etc.) with the requirements of the *evaluation report*. Are all relevant installation restrictions and requirements addressed? Are the inspection requirements correctly noted in the Statement of
Special Inspections? Are requirements on installer certification and anchor proof-loading or torque-testing noted where applicable? Is the anchor marking correct?

6 Necessary components for an evaluation

_Evaluation reports_ are generated on the basis of data submitted by the primary test laboratory retained by the manufacturer for the testing and assessment of the product and the anchor manufacturer. The submission to the evaluation agency may include but is not limited to the following elements:

1. Assessment report of product testing and assessment as issued by the primary testing laboratory.
2. Documentation of qualifications of report author(s), independence and accreditation of the testing laboratory and, as appropriate, the individuals conducting the assessment.
3. Proposed text parts, tables and figures.
4. Underlying test reports.
5. Documentation of random sampling of product for testing.
6. Quality control documentation.

The assessment report provided by the testing and assessment agency is central to the process. It provides a detailed description of the product/system, including the installation instructions, packaging, etc. and the fingerprinting tests conducted to enable future verification of product authenticity. It also describes the relationship between the test reports submitted and the requirements given in the applicable standard or acceptance criteria, and provides the essential recommendations for design values based on the assessment.

7 Critical concepts for the AHJ and EOR

It is important to understand that _evaluation reports_ are not code, nor are they compulsory. As noted in the introduction, _evaluation reports_ serve as evidence of code compliance for the AHJ. Of greater importance, however, is the reliance of the EOR (or other designers) on the contents of the report for the development of a structural design. The EOR is, as the title suggests, ultimately responsible for the safety of the design. Reliance on an _evaluation report_ for an anchor system or any other product relevant for structural safety does not relieve the EOR of this responsibility. On the other hand, it is virtually impossible for the EOR to verify the content of an _evaluation report_, e.g., by reviewing the underlying test reports. Aside from the time and effort that would be associated with such an activity, the test and assessment reports are considered proprietary and are typically not available for public review.

Given these facts, it is crucial that the process used by the evaluation agency to generate the _evaluation report_ contain sufficient safeguards to assure the integrity of the report. The following are requirements for evaluation agencies:
1. Accreditation per applicable ISO standards.
2. Maintenance of an internal peer-review process.
3. Maintenance of an open criteria development process.
4. Maintenance of a clear separation between reviewers and applicants.
5. Maintenance of reviewer competence (i.e., in the subject area of the evaluation report)

8 Conclusion

Anchors in concrete and masonry are critical safety-related elements in many structures. It is vital that they function in the manner intended to ensure the required level of structural safety.

Evaluation reports for proprietary construction products, including anchors, have grown to be essential elements in the chain of construction oversight, permitting, and inspection, and are indispensable to the designer, the building official and the manufacturer. Structural engineers (EORs) are particularly dependent on the accuracy and completeness of evaluation reports for anchors in concrete and masonry.

As such it is essential that evaluation reports be prepared with care and with complete independence. They should be read and understood by the EOR to ensure that all relevant information provided in the report be considered in the design and the installation of anchors. AHJs and their representatives should also have a thorough understanding of the contents of the applicable reports to ensure that the finished construction is in accordance with the intention of the construction documents. Failure to do so could result in a serious compromise of structural safety.

References:

2. ISO 17065:2012, Conformity assessment -- Requirements for bodies certifying products, processes and services, ISO, September 2012
QUALIFICATION OF A SYSTEM FOR POST-INSTALLED REINFORCING BARS UNDER THE RULES ESTABLISHED BY EOTA AND ICC-ES

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ABSTRACT

Systems for the installation of post-installed reinforcing bars using injectable polymer grouts are in widespread use in concrete construction. Their use extends from augmentation of existing structures to seismic retrofit applications. The qualification of such systems to verify their compliance with applicable building codes revolving around the concept of equivalence of performance between cast-in and post-installed reinforcing bars is currently conducted in Europe under provisions established by the European Organization of Technical Assessment (EOTA) and in the U.S. using criteria established by the ICC Evaluation Service, LLC (ICC-ES). Key issues include verifiable bond strength, viability of the adhesive delivery system, corrosion resistance, and response to tension loading under splitting-critical conditions where shear lag plays a significant role. In this paper, the rules established by these two organizations for the qualification of a post-installed reinforcing bar system are compared and discussed in detail. In particular, the background of the specific tests, the challenges associated with their conduct, the assessment procedure and recommendations for harmonization and improvement are discussed.

1 Introduction

Post-installed reinforcing bars consist of the installation of deformed reinforcing bars in holes drilled in concrete and filled with injectable polymer grouts. Post-installed reinforcing bars are generally used to construct concrete-to-concrete connections where new concrete is placed against existing concrete. As shown in Figure 1, the reinforcing bars are embedded in grout in a hole drilled into an existing concrete member and are cast in new concrete on the other side. By necessity, the portion of the reinforcing bars installed in existing concrete is straight, while the portion embedded in new concrete can be straight or hooked.

Post-installed reinforcing bars are often used to construct and extend existing reinforced concrete structural elements or to strengthen and rehabilitate them as shown in few examples in Figure 2.

In contrast to steel-to-concrete anchoring applications, concrete-to-concrete connections using post-installed reinforcing bars often involve relatively small edge and corner distances as dictated by cover requirements (in this case, concrete splitting governs the strength of the connection and is
anticipated in the expressions for required bar embedment). In addition, interface shear is resisted via concrete interlock (bars in tension, not shear), and the bars are typically embedded as required to develop the nominal tension yield strength of the reinforcing steel\textsuperscript{4,6}.

![Figure 1: Post-installed reinforcing bar (typ.)](image)

(a) Starter bars  
(b) Wall extension  
(c) Wall-to-slab connection

Figure 2: Extension of existing reinforced concrete structural elements using post-installed reinforcing bars

To allow the use of a post-installed reinforcing bar system for the construction of concrete-to-concrete connections, verification of the compatibility of the post-installed bars with neighboring cast-in bars in terms of strength, stiffness, and serviceability is required. In recent years, extensive research and testing\textsuperscript{10} has been conducted on this topic.

The load-displacement behavior of post-installed reinforcing bar systems is strongly linked to the adhesive performance and robustness in different installation conditions, such as temperature and humidity. Furthermore, the performance of post-installed reinforcing bars can be very sensitive to jobsite/installation conditions (e.g., improper or incomplete borehole cleaning, water penetration in the borehole before adhesive injection and/or reinforcing bar installation, corrosive environment), and loading conditions (e.g., freeze-thaw temperature cycles, sustained loading at high temperature, cyclic seismic loading). Also, in most cases, the performance of post-installed reinforcing bar systems is strongly affected by the type of drilling method, the quality and type of equipment used for installation, and the depth (or length) and diameter of the application.

The aforementioned considerations point to the necessity for appropriate post-installed reinforcing bar product qualification procedures\textsuperscript{1,2,3} with the primary goal of verifying that system performance, robustness, and installation are such that the behavior of a post-installed reinforcing bar can be considered comparable to that of cast-in reinforcing.
2 Principles of qualification of adhesive systems for use in post-installed reinforcing bar connections

Currently, the use of adhesive systems for the realization of connections with post-installed reinforcing bars is limited to qualified products according to the existing qualification guidelines (in Europe: EOTA TR 023\(^1\), to be replaced by the EAD 330087\(^2\); in USA according to AC308\(^3\)). The qualification procedures given in the existing guidelines are based on extensive research work carried out in past years, e.g. Spieth\(^8\) for static and Simons\(^11\) for seismic applications.

The basic principle for the qualification of any systems for establishment of post-installed reinforcing bar connections is the verification of comparable performance with cast-in reinforcing bars with respect to the failure modes anticipated in reinforcing bar design. These are: bond failure, understood as extraction of the bars from the concrete without significant concrete breakout, and splitting failure, which is typically characterized as cracking and spalling of the concrete cover. Bond failure occurs when sufficient confinement (e.g. large concrete cover) is provided to the bar, while splitting failure occurs when the concrete tensile resistance associated with the cover thickness is not sufficient to reach the load corresponding to bond failure.

A comparison in terms of load-displacement behavior (stiffness) based on the results of static tests conducted on anchorages and lap splices with different post-installed reinforcing bar systems and cast-in reinforcing bars has been conducted by Spieth\(^8\). These investigations indicate the importance of a qualification procedure to verify the compatibility of the bond performance of post-installed reinforcing bar systems with that of cast-in reinforcing bars. At that time, it was shown that the tested systems exhibited equivalent or superior bond strength to cast-in bars for cases where the controlling failure mode was concrete splitting, possibly due to the beneficial effect of the adhesive in redistributing the radial stress from the ribs of the bar into the surrounding concrete. These observations also agree with the results of other research projects in this field\(^12,13\). The systems tested by Spieth\(^8\) also exhibited stiffnesses similar to cast-in reinforcing bars.

This work also highlighted that, in addition to the assessment of the performance of a post-installed reinforcing bar system with respect to the relevant failure modes, the performance of the system vis-à-vis jobsite installation requirements as well as its sensitivity to a variety of serviceability conditions (corrosion, etc.) must be checked.

3 Qualification per EOTA TR 023 and EAD 330087

Based on research conducted mainly at Institute of Construction Materials (IWB) of the University of Stuttgart, EOTA TR 023\(^1\) was issued in 2006\(^1,2\). This document includes the provisions for the qualification of post-installed reinforcing bar systems to allow design of straight anchorages and lap splices per EN 1992-1-1\(^4\). Due to transitional procedures established by EOTA and the European Commission, a new EOTA EAD 330087\(^2\) has been issued to replace TR 023\(^1\) in the framework of the provisions of regulations (EU) 305/2011. The EAD 330087\(^2\) covers the qualification of post-installed reinforcing bar systems for static loading only (excludes fatigue and seismic actions) and contains no significant technical differences to the EOTA TR 023\(^1\). However, in EAD 330087\(^2\), qualification of systems for fire exposure has been introduced. This topic represents a significant
harmonization in the European context, since the qualification of post-installed reinforcing bar systems under fire exposure was previously assessed per national approval procedures (e.g. in France or Germany) based on non-standardized testing and assessment procedures, in some cases leading to different characteristic resistances for the same product. Furthermore, the results of this assessment provide the required input values to carry out a design under fire exposure as per EN 1992-1-2.

The test program can be divided in three parts:

1. Basic tension tests with confined test setup in uncracked concrete C20/C25 and C50/60 with the reinforcing bar sizes applied for to derive the average bond strength of the system.

2. Functioning tension tests intended to establish the product performance in different environmental and loading conditions, which may occur during the service life of the connection, as well as the viability of the adhesive delivery system and installation tools. These robustness tests include:

   - Installation with reduced borehole cleaning in dry and water saturated concrete
   - Installation at minimum temperature
   - Sensitivity to sustained load during freeze-thaw cycles and at elevated temperature
   - Sensitivity to installation direction
   - Installation at maximum embedment depth
   - Corrosion protection of the reinforcing bars
   - Durability in alkali and sulfurous environments

3. Optional tests to assess the resistance of the product to fire exposure are conducted as confined tension tests applying a sustained axial with increasing temperature. These tests allow to define a bond strength degradation curve as a function of temperature.

It is worth noting that no tests to verify the installation of bars close to an edge (e.g., at edge distances close to minimum cover) are prescribed. The minimum edge distance allowed for post-installed reinforcing bar applications per EAD 330087 is not system-dependent and is pre-established as a function of the drilling method (with and without the use of drilling aids) as shown Table 1. These values are significantly larger than the values recommended for concrete cover of cast-in reinforcing bars per EN 1992-1-1.

Table 1: Minimum edge distances for post-installed reinforcing bar systems according to EAD 330087.

<table>
<thead>
<tr>
<th>Drilling method</th>
<th>Bar diameter, $\phi$</th>
<th>Without drilling aid</th>
<th>With drilling aid</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hammer drilling, Diamond drilling</td>
<td>$&lt; 25$ mm</td>
<td>$30 mm + 0.06l_v \geq 2\phi$</td>
<td>$30 mm + 0.02l_v \geq 2\phi$</td>
</tr>
<tr>
<td></td>
<td>$\geq 25$ mm</td>
<td>$40 mm + 0.06l_v \geq 2\phi$</td>
<td>$40 mm + 0.02l_v \geq 2\phi$</td>
</tr>
<tr>
<td>Compressed air drilling</td>
<td>$&lt; 25$ mm</td>
<td>$50 mm + 0.08l_v$</td>
<td>$50 mm + 0.02l_v$</td>
</tr>
<tr>
<td></td>
<td>$\geq 25$ mm</td>
<td>$40 mm + 0.08l_v \geq 2\phi$</td>
<td>$40 mm + 0.02l_v \geq 2\phi$</td>
</tr>
</tbody>
</table>

$l_v = \text{embedment length}$

The average bond strength derived in the basic tension tests is multiplied by a reduction factor derived from the suitability tests and the results are compared with the minimum values $f_{hm}^{req}$.
according to Table 2. The rationale of the values $f_{bm}^{req}$ given in Table 2 were given in 15 (see (1) and (2)). However, the value $f_{bm}^{req}$ can be also calculated as per Model Code 20107 (see (3)).

1) $f_{bm} \approx 10$ MPa, according to a database of confined pullout tests C20/25 in uncracked concrete with cast-in reinforcing bars carried out at the University of Stuttgart (see 15).

2) Starting from the bond resistance design value $f_{bd}$, the average bond resistance $f_{bm}$ can be derived by multiplying $f_{bd}$ with the material partial factor $\gamma_c = 1.5$ and the ratio 1/0.75 to calculate the mean value from a given characteristic value assuming a coefficient of variation of 15% at 90% confidence for a high number of samples. However, the value $f_{bm}^{req}$ is (significantly) increased by a factor of approximately 2.2 to take into account the effect of cracked concrete.

3) The value $f_{bm} \approx 10.0$ MPa in C20/25 can be obtained starting from the value $f_{bd} = 2.3$ MPa and assuming that decrease of the bond strength with increasing embedment length is following a power law with an exponent of 0.55 (see Model Code 20107, Equation (6.1-19)). Considering that the tests according to the EAD 3300872 in C20/25 concrete should be conducted with an anchorage length equal to $10d_b$, and that for an anchorage far away from the edge Equation (8.10) of EN 1992-1-14 leads to an anchorage length of $40d_b$ to reach steel yielding, the factors to derive the average value of the bond strength listed in (2) are still valid. The derivation is shown in Equation (1).

$$f_{bm}^{req} = \gamma_c \cdot f_{bd} \left( \frac{40}{10} \right)^{0.55}$$

Table 2: Required bond resistances according to EOTA EAD 3300872

<table>
<thead>
<tr>
<th>Concrete strength class</th>
<th>Design values of ultimate bond resistance according to EN 1992-1-14 for good bond conditions $f_{bd}$ [N/mm²]</th>
<th>Required bond resistance for post-installed rebars $f_{bm}^{req}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C12/15</td>
<td>1.6</td>
<td>7.1</td>
</tr>
<tr>
<td>C16/20</td>
<td>2.0</td>
<td>8.6</td>
</tr>
<tr>
<td><strong>C20/25</strong></td>
<td><strong>2.3</strong></td>
<td><strong>10.0</strong></td>
</tr>
<tr>
<td>C25/30</td>
<td>2.7</td>
<td>11.6</td>
</tr>
<tr>
<td>C30/37</td>
<td>3.0</td>
<td>13.1</td>
</tr>
<tr>
<td>C35/45</td>
<td>3.4</td>
<td>14.5</td>
</tr>
<tr>
<td>C40/50</td>
<td>3.7</td>
<td>15.9</td>
</tr>
<tr>
<td>C45/55</td>
<td>4.0</td>
<td>17.2</td>
</tr>
<tr>
<td><strong>C50/60</strong></td>
<td><strong>4.3</strong></td>
<td><strong>18.4</strong></td>
</tr>
</tbody>
</table>

According to EAD 3300872, the average bond strength (as derived from the pullout strength as measured on shallow embedments and assuming uniform bond) of the tested post-installed reinforcing bar system must fulfill the requirements of Table 2 to allow the design of anchorages and lap splices according to EN 1992-1-14. However, the minimum requirement for qualification is to reach at least the value $f_{bm} = 7.1$ MPa for all concrete strength classes C20/25 to C50/60 according to Table 2. For products exhibiting an average bond strength lower than those given in Table 2 (but at least $f_{bm} = 7.1$ MPa) for a given concrete strength class, a bond strength reduction factor, $k_b \leq 1.0$, is
provided in the European Technical Assessment (ETA). In this case, a bond strength lower than the one recommended by EN 1992-1-1\textsuperscript{4} is obtained and reported in the European Technical Assessment (ETA). Note that the assessment thus assumes that increasing bar embedment in this manner will compensate for the reduced system performance as compared with cast-in bars. This supposition has not, to the knowledge of the authors, been verified in practice.

The performance of post-installed reinforcing bars in cracked concrete is also checked and the maximum allowed pullout strength decrease in the case of a longitudinal crack of 0.3 mm width is 25%. If a post-installed reinforcing bar system does not fulfill this requirement, the anchorage length must be increased 50% to reduce the probability that the post-installed reinforcing bar is affected by a longitudinal crack along its entire length. Alternatively, it is permitted to waive the requirement on cracked concrete performance if the pullout strength in uncracked concrete significantly exceeds the minimum requirement shown in Table 2 (e.g. in C20/25 $f_{hm} \geq 15.0$ MPa).

It is noted that the requirements in cracked concrete were introduced under the assumption that the behavior of post-installed systems would exhibit more sensitivity to cracked concrete compared to equivalent cast-in reinforcing bars.

Summarizing, the assessment procedure for post-installed reinforcing bar system following the requirements of EAD 330087\textsuperscript{2} targets the equivalency with cast-in reinforcing bars in terms of load-displacement behavior (ignoring potential differences in stiffness of the systems) and, importantly, permits qualification for systems with reduced performance by assigning a reduced bond value/increased embedment requirement.

4 Testing and assessment per AC308

In 2013, on the recommendation of the Concrete Anchor Manufacturers Association (CAMA), ICC-ES adopted amendments to the document AC308\textsuperscript{3} a) to incorporate the qualification and design provisions of ACI 355.4 and ACI 318-11, respectively, and b) to add testing and assessment requirements to qualify adhesive systems for post-installed reinforcing bar applications. The qualification procedures in AC308 were based on those developed for the EOTA TR 023\textsuperscript{1} and EOTA EAD 330087\textsuperscript{2}, but with the inclusion of (1) an additional test series intended to verify the near-edge performance of the system at relatively deep embedment depths (the so-called, bond/splitting behavior test developed by CAMA), and (2) seismic cyclic tension tests. However, AC308 does not permit the qualification of systems that do not meet the minimum strength and stiffness requirements established in the criteria, and qualified systems are assumed to be designed using the provisions of the ACI code applicable to cast-in straight bars.

AC308\textsuperscript{3} addresses the assessment and design of post-installed reinforcing bar adhesive systems proportioned “according to the development and splicing provisions of straight reinforcement” as given in ACI 318\textsuperscript{6} and does not limit the applications which are covered (e.g., starter bars rather than beam-column connections, etc.). Therefore, it may be assumed that the all applications suitable for cast-in straight bars are covered by the criteria, including non-contact lap splices, starter bars, and shear dowels.
The test program for post-installed adhesive reinforcing bar qualification is outlined in Table 3.8 of AC308\(^3\) and requires performing tests to establish the suitability of the system for post-installed reinforcing bar applications in terms of:

1. Bond strength.

These tests are intended to determine the baseline bond strength for product evaluation and assessment. Tests are performed in uncracked, low and high strength concrete, and with different reinforcing bar sizes. The tension tests are performed in confined setup and the required embedment depth is equal to 7 times the reinforcing bar diameter, \(d_b\).

2. The ability of the system to develop bond resistance over the embedded length when splitting is the connection controlling failure mode (Figure 3).

This pass-fail test, where the splitting behavior of post-installed reinforcing bars is compared side-by-side with that of cast-in reinforcing, is extremely important. If the post-installed adhesive system is too stiff, excessive shear lag could lead to “zipper-like” failure of a corner or near-edge bar. If the adhesive is too “soft”, relaxation of the post-installed bar may lead to excessive displacements and opening of the joint between existing and new concrete, leading to corrosion and loss of strength (e.g., shear transfer).

![Figure 3: Bond/splitting behavior test (AC308\(^3\))](image)

3. Sensitivity of the product performance in different hole cleaning conditions, freeze-thaw temperature cycles, exposure to high alkalinity/sulfur environments, installation orientation, etc.

These tests are intended to verify the adhesive system reliability.

4. Ability of the system to be installed in deep holes and in different temperatures without compromising its performance (e.g., air voids).

These tests are performed in concrete and in clear rigid tubes and require installation of the system at 60 times the maximum reinforcing bar diameter, \(d_{b,\text{max}}\), for which recognition is sought. Installation shall be performed at maximum installation temperature for concrete and standard temperature for the clear tubing. For the former, tests need to be performed at the minimum
concrete cover for which the system is being qualified. The minimum edge distance allowed for post-installed reinforcing bar applications, although not strictly product dependent, is established in these tests and can be reported in the product evaluation report (ICC-ES ESR) as a recommended value. The minimum concrete cover recommended by ACI 318 for cast-in reinforcing bars governs. Ultimately, it is responsibility of the designer and installer to verify the feasibility of the installation and the quality/type of equipment needed for proper hole drilling and cleaning, and for reinforcing bar installation (e.g., means and methods).

5. Ability of the system to provide corrosion resistance comparable to cast-in reinforcing bars.

These tests are intended to verify that the system provides comparable protection against corrosion under more aggressive environmental conditions.

6. Ability of the system to perform adequately (e.g., as cast-in reinforcing bars) under seismic cyclic tension conditions

These tests procedures are intended to verify hysteretic behavior that is comparable with benchmark investigations into the cyclic response of cast-in reinforcing bars conducted at UC Berkeley in the late 1970s (Eligehausen et al.16). The test setup is derived from investigations of post-installed reinforcing bars conducted by Simons11. Qualification of the system is dependent on both bond strength and bond strength degradation targets being met under a constant cyclic slip protocol (Figure 4).

![Hysteretic behavior of cast-in reinforcing bars (typ) and comparison of bond strength degradation of cast-in vs. post-installed reinforcing bars](image)

Figure 4: Cyclic load protocol and assessment for post-installed reinforcing bar seismic qualification

The assessment of a post-installed adhesive system in accordance with ICC-ES AC3083 consists in the following:

- The average bond strength of the system in low and high strength, uncracked concrete conditions, considering all applicable reduction factors derived from reliability and cyclic tension tests, must equal or exceed 7.5 MPa and 11.8 MPa, respectively.

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The values 7.5 MPa and 11.8 MPa were established by equating the reinforcing bar steel yield strength to the pullout strength of a reinforcing bar embedded its full development length as calculated in accordance with ACI 318, Chapter 25 (see Eqs. (2) to (4)). The average bond strengths set as requirements by ACI 308 are obtained assuming no edge influences (e.g., $(c_b+K_{tr})/d_b = 2.5$), $\psi_t = \psi_e = 1.0$, and $\psi_s = 0.8$. Furthermore, the material partial factor $\phi = 0.65$ and the ratio $1/0.75$ (needed to obtain a mean value from a characteristic value assuming a coefficient of variation of 15% with a confidence level of 90%) are taken into account.

$$\tau_{\text{bond}} \cdot \pi \cdot d_b \cdot l_d = A_f \cdot f_y = \frac{\pi \cdot d_b^2}{4} f_y$$

$$l_d = \left[ \frac{3 f_y}{40 \sqrt{f_c} c_b + K_{tr}} \right] (\text{lb, in.})$$

$$\tau_{\text{bond}} = 3.33 \cdot \sqrt{f_y} \left( \frac{c_b + K_{tr}}{d_b} \right) \left( \frac{1}{\psi_t \cdot \psi_e \cdot \psi_s} \right) (\text{lb, in.})$$

Where: $f_y$ = steel yielding strength; $f_c$ = concrete compressive strength; $\lambda$ = factor for lightweight concrete; $d_b$ = bar diameter; $c_b$ = minimum edge distance; $K_{tr}$ = confinement term; $\psi_t$, $\psi_e$, and $\psi_s$ = factors for top reinforcement, epoxy coating and bar size, respectively, and $(c_b+K_{tr})/d_b \leq 2.5$.

- As noted earlier, failure to achieve the specified bond strengths and/or to meet the strength degradation parameters is cause for non-qualification of the system.

- In case the system has not been previously qualified for cracked concrete conditions, tests in cracked concrete with #4 reinforcing bars must be performed and requirements on bond strength (average bond strength at failure, $\tau_{u,m,cracked} \geq 0.5 \cdot \tau_{u,m,uncracked}$) and displacement (average displacement at failure, $\delta_{u,m,cracked} \leq (\delta_{u,m,uncracked} + 0.5 \text{ mm})$) must be met (Note that this requirement is based on Simons; see Figure 5b). Tests are required both in low and high strength concrete. Failure to meet these requirements is cause for non-qualification of the system.

For qualified systems, design is permitted in accordance with the development and splicing provisions for straight reinforcing as given in ACI 318. For systems that do not fulfill all requirements in high strength concrete, development length is calculated using the concrete compressive strength corresponding to the low end of the low-strength concrete strength range ($f_c' = 17.2 \text{ MPa} / 2,500 \text{ psi}$), regardless of the actual design concrete strength. For systems that do not fulfill the requirements for cyclic loading, applications are limited to structures assigned to Seismic Design Categories A and B (essentially non-seismic).

5 Comparison of qualifications according to EOTA and AC308

Although the testing and assessment procedures currently in place in Europe and the U.S. were developed on the basis of the same research background and are similar in content, they contain significant differences.
Permissible applications for a post-installed reinforcing bar system qualified according to EAD 330087\textsuperscript{2} differs from the assumed application range associated with AC308\textsuperscript{3} as shown in Table 3.

Table 3: Field of application for post-installed systems qualified according to EAD 330087\textsuperscript{2} and to AC308\textsuperscript{3}

<table>
<thead>
<tr>
<th>Option</th>
<th>EAD 330087\textsuperscript{2}</th>
<th>AC308\textsuperscript{3}</th>
</tr>
</thead>
<tbody>
<tr>
<td>Static loading</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>Seismic loading</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>Fatigue Loading</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>Fire exposure</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Limitations on</td>
<td>Larger cover than cast-in bars according to EN 1992-1-1\textsuperscript{4}</td>
<td>If verified by test, same cover as cast-in bars according to ACI 318\textsuperscript{6}</td>
</tr>
<tr>
<td>concrete cover</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

EAD 330087\textsuperscript{2} provides schematic representations of the post-installed reinforcing bar configurations addressed by the standard. These distinguish between cases where the concrete is required to be in tension and where, theoretically, it is not. Thus, compression on a column to foundation connection with straight bars is permitted whereas overturning on the same column is not unless special design procedures (strut and tie, etc.) are employed to verify the concrete resistance. One of the cases shown, a beam designed to have pinned supports thereby avoiding the introduction of negative moment at the joint, is particularly misleading since this case rarely, if ever, occurs in practice. In addition, other configurations that could be permissible are not shown. On the other hand, AC308\textsuperscript{3} links the qualification and the design procedures to all cases where post-installed reinforcing bars are proportioned according to the development and splicing provisions of straight reinforcement of ACI 318\textsuperscript{6}.

The two documents adopt the same criteria to assess the reliability of the system and its capability to provide protection against reinforcing bar corrosion, but deviate in the way the adhesive system average bond strength is assessed. EAD 330087\textsuperscript{2} imposes a limit to the average bond strength of the product to \(f_{bm}^{req}\) according to Table 2, while AC308\textsuperscript{3} requires reaching a minimum of 7.5 MPa for 2,500 psi concrete (17.2 MPa) or 11.8 for 6,500 psi concrete (44.8 MPa). It is noted that in the European framework, in the case that the basic bond strength requirement above is not fulfilled, a system can still be qualified as long as its bond strength is at least equal to \(f_{bm} = 7.1\) MPa for all concrete strength classes C20/25 to C50/60 (ref. Table 2 and reduction factor \(k_b\) in Section 3). The same concession is not allowed in AC308\textsuperscript{3}, meaning that a system that does not reach the required minimum values cannot be issued an evaluation report.

As mentioned above, AC308\textsuperscript{3} includes an additional pass/fail test (bond/splitting behavior), which is quite demanding and stringent, and seismic qualification tests. Testing procedures and requirements for fire loading conditions are, however, not required and are contemplated only by EAD 330087\textsuperscript{2}.

Finally, the two criteria present different options to deal with cracked concrete conditions. In the European case, testing is only optional and its scope is solely to avoid using longer development lengths (factor \(\alpha_{lb}\), as described in Section 3). The system is otherwise always qualified. According to AC308\textsuperscript{3}, however, testing in cracked concrete is mandatory for systems not otherwise qualified for
cracked and uncracked concrete post-installed adhesive anchor applications, and in case the requirements are not fulfilled, the systems cannot be qualified.

In this specific case, EAD 330087\(^2\) and AC308\(^3\) assume a different load carrying behavior of cast-in reinforcing bars in cracked concrete conditions. The approach of EAD 330087\(^2\) is, in fact, based on the results of Eibl et al.\(^{14}\), where a cast-in reinforcing bar bond strength decrease of approximately 25% was observed in 0.3 mm longitudinal cracks. On the other hand, AC308\(^3\) takes into account the results of Simons\(^{11}\), which indicated that the decrease in bond strength of cast-in reinforcing bars can be as high as 50% for a crack width of 0.4 mm.

![Figure 5: Influence of longitudinal crack width on the bond strength of cast-in reinforcing bars.](image)

Finally, some significant differences between systems qualified according to EAD 330087\(^2\) and AC308\(^3\) are related to their installation conditions and restrictions. EAD 330087\(^2\) prescribes minimum edge distances according to Table 1, while the limitations for post-installed reinforcing bar systems qualified according to AC308\(^3\) are the ones applicable to cast-in reinforcing bars as per ACI 318\(^6\). Furthermore, the assessment according to AC308\(^3\) requires successful installations at a given embedment depth (60\(d_b\)), while EAD 330087\(^2\) allows testing to verify the maximum reachable embedment depth (e.g., 20\(d_b\), 80\(d_b\), etc.), allowing more flexibility.

### 6 Conclusions

The qualification procedures of post-installed reinforcing bar systems to verify their compliance with applicable building codes according to EAD 330087\(^2\) and AC308\(^3\) present similarities in the way testing and assessment are carried out. As shown in Table 3, however, the range of applications addressed varies significantly.

However, while both guidelines provide safeguards to restrict post-installed reinforcing bar systems that exhibit very low stiffness, EAD 330087\(^2\) does not include verification of whether a system is overly stiff. Since the development of new bonding materials to address e.g., fire-resistance, may result in systems with significant differences in the load-displacement response, a bond/splitting test to address shear lag for long embedments as per AC308\(^3\) is advisable and should be added to the European guidelines.
Finally, the fact that a product or system could be qualified in accordance with EAD 330087\textsuperscript{2} even when its performance is lower compared to equivalent cast-in reinforcing bars and independently of its performance in cracked concrete conditions appears to be potentially unconservative (e.g. non-splicing applications).

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SYSTEMS WITHOUT ACTIVE DRILL HOLE CLEANING / INTEGRATION OF HOLLOW DRILL BITS INTO ETAS

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ABSTRACT

The reduction of dust emission becomes more and more important, as safety and health protection on site gets into focus. This is in contradiction to most current post-installed anchors, which require a properly cleaned drill hole.

That leads into the development of post-installed anchors, which avoid dust emission during installation. We know two approaches: 1. Conventional systems for which drilling with simultaneous cleaning – either with external dust extraction or with so-called hollow drill bits – is used. 2. Systems, which can be installed in conventionally hammer-drilled holes, but which do not require an active drill hole cleaning – those systems clean their drill holes during installation and do not need any cleaning tools.

The second approach came up in the last few years and gives the installer beside the safety and health protection on site additional benefit. The installation is faster, cheaper and easier and follows the principle of Poka-Yoke.

If we talk about high performance steel anchors and bonded anchors, we always have to think about European Technical Assessments or other approvals, as well. Certainly, also systems without active drill hole cleaning and systems, which use drilling with simultaneous cleaning by hollow drill bits, have to be evaluated properly. We realise that additional parameters for the evaluation of those innovative systems have to be observed, to guarantee a proper function. This essay gives an overview.

1 Introduction

Working on a building site means dealing with dust. The drill dust can occur from different sources. In the fastening technology most of the dust is produced while drilling and cleaning drill holes. Drill dust is very fine-grained and can cause serious diseases, if it is inhaled continuously.

Physicians, authorities, the industry and last but not least more and more craftsmen on site are aware of this serious problem and start to act to improve the occupational safety on building sites with regards to drill dust emission.
In contradiction to that most of the current post-installed anchors in the market require a proper drill hole cleaning. The drill hole cleaning ensures a reliable function of the anchor even under building site conditions and is a requirement to enable the load resistances, which are determined in approvals or assessments. As shown in figure 1 for traditional anchors there is a strong relation between drill hole cleaning and corresponding performance.

![Figure 1: Influence of drill hole cleaning on load performance](image)

It is not surprising, that the drill hole cleaning has the biggest impact on bonded anchors. But also the performance of mechanical anchors can be influenced by insufficient drill hole cleaning.

## 2 Traditional anchors

### 2.1 Mechanical Anchors

Sleeve anchors and wedge anchors belong to the group of mechanical anchors (see figure 2).

![Figure 2: Sleeve anchor (top) and wedge anchor (bottom)](image)
Those kinds of anchors have an expansion sleeve or an expansion clip. By accident not removed drill dust can get between cone bolt/cone nut and expansion clip/sleeve and impede a proper interaction of the two components. Again, massive presence of drill dust can reduce the free space in the drill hole or an layer of drill dust on the surface of the drill hole wall can reduce the free cross section of the drill. Both of the above mentioned situations can make the installation difficult of even impossible. Most of the current mechanical anchors require drill hole cleaning (see figure 3).

For sake of completeness, it has to be mentioned that also for plastic plugs like e.g. frame fixings the presence of drill dust can have influence on their mountability.

2.2 Bonded Anchors

Traditional bonded anchors (see figure 4) are very sensitive to the lack of drill hole cleaning. In addition to the influence on the mountability, the drill creates a separating layer between substrate and mortar shell.

This layer reduces the bond strength and often the remaining load performance is below 50 % of the value in a proper cleaned hole.
The required cleaning procedure is defined by the manufacturer of the chemical mortar (example see in figure 5) and can be different for various systems. It even occurs that the same product has different cleaning procedures depending on the related performance.

For the installers, who work with different products – maybe even from different brands – this situation is very annoying and risky, as well, as products and cleaning procedures get mixed up.

Again, the cleaning procedure take quite a long time – about 25% of the installation time is cleaning – and the installer has to keep various cleaning tools, like e.g. brush and blow out pump, at hand.

3 Installation of traditional anchors using hollow drill bits

As cleaning of drill holes takes its time and regulations for dust emission are very strict in some countries, the anchor manufacturers more and more evaluate, if their fasteners show the same performance in drill holes, which are made with hollow drill bits and no further cleaning, compared to traditional installation methods. Certainly, the cleaning of the drill hole has not the same significance for mechanical anchors like for bonded ones, but this is reflected in the assessments of the products.

3.1 General

Hollow drill bits can be used instead of hammer drill bits states PGM Prüfgemeinschaft Mauerbohrer (PGM Masonry Drill Bit Certification Board) on their web site. But the drill hole cleaning cannot be omitted automatically without showing the comparability to the traditional cleaning.

If the traditional drill hole cleaning should be replaced, the corresponding tests with hollow drill bits have to show the equivalence to the traditional cleaning procedure. The tests of hollow drill bit have to be done with the connected dust extraction system. The suitability of the dust extraction system in combination with the particular hollow drill bit has to be proved.

3.2 Mechanical anchors

As mentioned earlier, for mechanical anchors drill dust can influence the function and can lead to a loss of performance. But as they are not so sensitive concerning intensive cleaning, it is sufficient to remove the loose drill dust. Instead of traditional blowing out a hollow drill bit can be used to get off the drill dust. Many European Technical Assessments (ETA) show, that this cleaning method is approved, but that it is not necessary to define the further details like the type of hollow drill bit, etc.
3.3 Bonded anchors
For bonded anchors the detailed definition of approved hollow drill bits has much more relevance.

The reason is that not all hollow drill bits in the market have the same cleaning performance and every bonded anchor reacts different on the available hollow drill bits. Even within the range of hollow drill bits of the same brand the cleaning performance of the different diameters can vary.

The result is that every single product has to be tested in combination with every hollow drill bit (family), which shall be taken into the ETA or respectively other assessments.

Similar to the testing of a bonded anchor with traditional drill hole cleaning, also for the use with a hollow drill bit different installation conditions have checked. Often the installation in damp substrate are decisive, as the drill dust clumps together and sticks to the drill hole wall.

3.4 Specific rules for the use of hollow drill bite to replace traditional cleaning
There are some differences in the installation procedure if the installer changes from standard hammer drilling to drilling with a hollow drill bit.

Certainly, the installer can use the same power tool for drilling as the hollow drill bits have the same connecting adapter like standard hammer drill bits.

But in addition hollow drill bits have to be connected to a dust extraction system, which has to meet the performance class mentioned in the particular ETA (or other assessment) of the anchor. Before drilling the dust extraction system has to be turned on and its effectiveness has to be ensured. It is recommenced to check the proper function of the dust extraction system at least before and after drilling each single hole (see figure 6). Again, even if it sounds trivial: The dust extraction device has to run on full power and its components like e.g. the filter, the hose, etc. may not be blocked.

![Functional test of dust extraction system with hollow drill bit](image)

Figure 6: Functional test of dust extraction system with hollow drill bit

4 Systems without active drill hole cleaning
The latest trend in the fastening technology is to make the installation faster, easier and cheaper. This means, that the installation follows a simple rule – the less steps the better. Because less steps means less source of errors and this is the principle of Poka-Yoke.
Especially the drill hole cleaning belongs to the steps during installation, which is often omitted maybe to safe time or maybe as its importance is underestimated – both leads to imperiled constructions.

In the market more and more systems are available, which do not require any active drill hole cleaning. No active drill hole cleaning means that the system cleans the drill hole by itself as much as necessary for a proper function.

4.1 Mechanical anchors
Astonishingly, up to now only a few mechanical anchors are approved for the use in not cleaned drill holes. For sure the demand from the market for mechanical anchors without drill hole cleaning is clearly lower than for other systems, like e.g. bonded anchors. The reason might be that the general opinion is that the necessity for mechanical systems is lower. However it was mentioned earlier that also mechanical anchors can be affected by insufficient drill hole cleaning.

The omission of drill hole cleaning can be included into assessments or approvals without efforts for systems for overhead installation as well as mechanical anchors, which are installed vertically into the ground, if the drill hole depth has been increased to give the remaining drill dust enough space. For horizontal installed mechanical anchors up to now no approved system offers the possibility to omit the drill hole cleaning.

4.2 Bonded anchors
Within the bonded anchors it has to be differred between capsule systems and injection systems. For both types solutions to omit drill hole cleaning are available, but the solutions differ.

4.2.1 Injection systems
Drill dust creates a separating layer between substrate and mortar shell. But the current systems are not able to establish a bond strength that results in a reasonable load performance.

For this reason bonded expansion anchors are used for injection system without cleaning. The principle of function is not trusting on the bond only, but also on expansion forces due to the coned shape of the anchor rod (example see figure7).

![Figure 7: Example for an anchor rod with coned shape](image)

But even the bonded expansion anchor has to prepare the drill hole itself. The drill dust has to be compressed at the end of the drill hole to give the anchor enough space to embed. When indicated, the drill hole depth has to be increased to ensure that the required embedment depth can be observed.
4.2.2 Capsule systems
Capsule systems clean their drill holes during installation, as the threaded rod is driven into the substrate with a power tool with rotary and hammer action. Fragments of the glass capsule and the sand scratch along the drill hole wall and clean up the drill hole like that (an example for a capsule system without active cleaning is given in figure 8).

![Figure 8: Example for a system without active drill hole cleaning](image)

Today there are on the one hand capsule systems with cone shaped anchor rods but on the other hand also systems with conventional threaded rods with oblique tip available in the market (see examples in figure 9).

![Figure 9: Capsule system with con shaped anchor rod (left) and capsule system with conventional threaded rods with oblique tip (right)](image)

As for capsule systems the cleaning is done automatically during installation and additional step like drill dust compression is required, modern capsule systems meet the target best and fulfill the principle of Poka-Yoke already today.

5 Conclusion
For traditional anchors drill hole cleaning is a must, unless the use of hollow drill bits is approved. The next generation of anchors do not require active drill hole cleaning, which means that the drill hole is cleaned by the anchor itself during installation. Especially within the bonded anchors systems without active drill hole cleaning are available.

Currently the number of post-installed anchors, which are approved with hollow drill bits or which even function without active drill hole cleaning are rare, but it is assumed that the number of those systems will increase dramatically within the next years, until it is a standard in the market.

References:
CAST-IN ANCHORS
INFLUENCE OF EDGE BEARING ON THE FRONT-EDGE SHEAR BREAKOUT CAPACITY

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ABSTRACT

Edge bearing stresses on shear-loaded anchorages can enhance the shear capacity because of the confining stresses provided by the compressive bearing stress. However, too much bearing stress can have a negative influence on the shear breakout strength; the higher bearing stress cracks the concrete in a pattern similar to the failure from a pure bearing load, compromising the shear breakout when the anchorage is near an edge. A pilot test program of 11 tests was conducted to examine this effect; this work was initiated from an investigation of a precast wall panel anchorage failure (gravity and wind connection), where the bearing stress on an embedded anchorage likely reduced the concrete shear breakout capacity when the anchorage was near the free edge of the concrete member.

The ACI 318-14 Building Code, Chapter 17 recognizes the interaction of tension and shear on a connection, yet the interaction between bearing (compression) and shear does not appear to have ever been studied experimentally. This pilot test program examined plain steel plates (plates resting on the top of the concrete with a thin layer of grout beneath the plate) loaded in compression and plates with headed studs embedded in concrete loaded in compression with the compression load near an edge. The test specimens were not limited by any lateral side boundary conditions. This paper examines the second phase of testing where headed-stud connections were loaded toward a free edge with an orthogonally applied edge-bearing load. Bearing stress ranged from 0 to 16.4 MPa (2,375 psi) and the front-edge distance to the studs ranged from 51 to 76 mm (2 to 3 in.). One edge of the anchorage plate in bearing was located flush with the edge of the concrete. Concrete cover requirements were maintained on the embedded anchors.

The test findings showed that the shear capacity of the embedded anchorage increased as the bearing stress increased. However, above a bearing stress of 17.2 MPa (2,500 psi), the influence of bearing stress was adverse and reduced the shear breakout capacity computed by ACI 318 methods. A modification relationship to account for this affect is proposed.

1 Introduction

In investigating the cause of a precast concrete panel falling off a hotel facade in the Midwest United States, we uncovered an interesting behavior of the panel connection. The panel-to-structure connection, shown in Figures 1 and 2, applied the panel gravity load to a spandrel beam. The connection consisted of an angle bearing on a steel plate, attached to the top horizontal surface of a
spandrel beam, with expansion or post-installed concrete anchors. The anchors were installed at a point atop the spandrel beam, outside the reinforcement cage core. Our investigation of the connection shear capacity revealed a reduced shear capacity due to the gravity load bearing on the connection plate attached to the spandrel.

Other conditions like the one shown in Figures 1 and 2 exist when precast elements bear on supporting members. Two simple examples are the precast tee stems bearing on a ledger beam, Figure 3, and an inverted-tee beam or a stem of a precast tee beam bearing on a corbel, as sketched in Figure 4. In each of these conditions, the connection has a shear force applied toward a free edge due to time-dependent shrinkage or temperature deformations.

Figure 1: Cross section of cladding panel supported by a spandrel beam

Figure 2: Loads on embedded anchorages in top of spandrel; see middle connection in Figure 1. 
(C = Compression or bearing force, V = Shear)
2 Past Research

In 1963, Kriz and Raths investigated the bearing strength of column heads at the Portland Cement Association (PCA) Laboratories\(^1\). Their testing resulted in a bearing strength relationship for finite-width column heads, which can be found as Equation 5-54 in the PCI Handbook - 7th Edition\(^2\). The PCA investigators noted that “the bearing strength of column heads is reduced considerably when outward horizontal forces (\(N_u\)) are applied in combination with the vertical loads.” This conclusion can be carried over to the shear strength of a headed stud anchorages near an edge if affected by a concentrated compressive force. As shown in the PCI equation below, Equation 1 (English Units), it is difficult to determine the shear strength of a headed stud connection if written in this form. \(V_n\) is the vertical bearing strength, \(V_n\) is the bearing load and \(N_u\) is normal to the vertical load, that is, the shear load. In addition, the PCA research did not use headed studs to anchor the column head bearing plate. The steel bearing only rested on the column head concrete.

\[
\phi V_n = \phi \left( \frac{SW}{200} \right)^{\frac{N_u}{V_n}} \left( 0.85f'_{cL}A_1 \right) \left( \frac{A_2}{A_1} \right) \leq 1.1f'_{cL}A_1 \quad (1)
\]

Based on the hotel facade investigation and 1963 PCA research, we believe a combined compression-shear interaction relationship exists when concrete anchors are located within 3 to 5 anchor diameters from a concrete edge; this represents a front-edge distance within the side concrete cover. The edge distance represented in the PCA equation is an arbitrary distance whereby any lateral reinforcement in the member would be ineffective. The PCA testing focused on the bearing capacity, \(V_n\), of a precast concrete column head. This paper inverts the focus and examines the shear strength of a headed-stud connection under the presence of an axial compressive load. Moreover, no lateral side edges will be present to limit the transverse width of the concrete breakout surface. The testing reported herein applies to any bearing seat anchored by headed studs.
3 Experimental Program

An experimental program was performed where a shear-loaded anchorage located close to an edge was subject to combined shear and bearing loads. A previous pilot testing program with bearing-only loads was conducted prior to this testing. We found the edge-bearing capacity of concrete is quite high, with bearing loads about 10 times greater than the predicted shear load. Thus, we selected realistic, in-service bearing stresses for these test anchorages, and theorized the bearing influence on shear capacity would be a multiplicative factor, instead of an interaction relationship.

3.1 Test Specimens

The anchorages were located at the edge of a 1.5 x 3.0 x 0.4 m (5 ft x 10 ft x 1 ft-3 in.) deep concrete specimen used for pryout testing for another experimental study. Tests were conducted by restraining the movement of the specimen. The simple loading apparatus and test slab is shown in Figure 5.

Front-edge distance to the headed studs was 51 or 76 mm (2 or 3 in.), and the, s1, anchor spacing was either 89 or 114 mm (3.5 or 4.5 in.). All tests used a two-stud anchorage configuration in the near-edge location. Headed studs were 12.7 mm (1/2 in.) in diameter with an effective depth, h_{ef}, of 81 mm (3.2 in.). The h_{ef} / d_a ratio was a constant 6.4. The ultimate tensile strength of the headed studs was 536 MPa (77.7 ksi). All load-deformation curves for the headed studs tested in direct tension in air exhibited rounded, but ductile, behavior. Steel failures did not govern any of the testing reported herein.

The steel plates used were 12.7 mm (1/2 in.) thick. Plate sizes are shown in Table 1 with a calculated bearing contact area of 15,490 or 20,650 sq. mm (24 or 32 sq. in.). The right-hand column of Table 1 shows the three plate configurations used. The plates had extensions that projected beyond the slab edge. As shown, the plate extension had a hole for a bolt and clevis assembly for the shear-load application.

Table 1 – Steel plate dimensions for the anchorage tests

<table>
<thead>
<tr>
<th>Test Plate</th>
<th>Test Geometry</th>
<th>Brg Plate Size</th>
<th>Bearing Area (mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PL-7A</td>
<td>c_{at} (mm)</td>
<td>s_1 (mm)</td>
<td>x (mm)</td>
</tr>
<tr>
<td>PL-7B</td>
<td>50.8</td>
<td>88.9</td>
<td>152.4</td>
</tr>
<tr>
<td>PL-8A</td>
<td>76.2</td>
<td>88.9</td>
<td>152.4</td>
</tr>
<tr>
<td>PL-7A</td>
<td>50.8</td>
<td>114.3</td>
<td>203.2</td>
</tr>
</tbody>
</table>

All edge plates were positioned on the form bottom so 0.4 m (1 ft - 3 in.) of concrete was placed above. This ensured good consolidation around the headed studs and thus trapped air voids beneath the bearing plate were eliminated. The slabs were reinforced with a nominal amount of welded wire reinforcement (WWR) for handling purposes; where applicable, the mesh was cut away in the
vicinity of the stud anchorages to avoid any possible contribution of reinforcement to the measured test load.

3.2 Concrete Properties

The concrete slab was cast with a nominal 41.5 MPa (6,000 psi) normal weight concrete mixture containing 12.7 mm (1/2 in.) angular, granite gravel, and no air entrainment. The concrete properties are presented in Table 2 for the tests. We started testing the anchorages at a concrete age of 23 days and finished at 45 days. Concrete cylinder compressive strengths were obtained at the beginning and end of testing, and at 28 days. The static modulus of elasticity was tested at the same time as the concrete compressive strength. Tensile split-cylinder tests were also performed at the beginning and end of testing. The tensile strengths obtained were consistent with values expected. The concrete strength was not a variable in the experimental program.

Table 2 – Concrete properties for the tests

<table>
<thead>
<tr>
<th>Concrete Age (days)</th>
<th>Concrete Average Values (150 x 300 mm cylinders)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$f'_c$ (MPa)</td>
<td>Static Modulus, $E$ (MPa)</td>
</tr>
<tr>
<td>14</td>
<td>37.2</td>
<td>-</td>
</tr>
<tr>
<td>23</td>
<td>40.3</td>
<td>28,020</td>
</tr>
<tr>
<td>28</td>
<td>40.8</td>
<td>29,070</td>
</tr>
<tr>
<td>45</td>
<td>43.4</td>
<td>28,730</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
- Col (2) Compressive strength based on the average of three (3), 150 x 300 mm cylinders.
- Col (5) where $f_{sp} = n (f'_c)^{0.5}$
- Concrete unit weight = 2,420 kg / cubic meter (normal weight)

SI Units: 1,000 psi = 6.895 MPa, 1 pcf = 16.026 kg / cubic meter

3.3 Test Procedures

Figure 5 shows the test set-up. Each slab was tested flat, and the slab was rotated in-plane to access other embedded edge anchorages located on the sides. A test frame was built above the slab and tied down to the test floor for the bearing reaction. The shear test frame reacted near the ends of the slab edge, as to be located outside the potential breakout cone that forms along the long-edge direction of the test specimen. Test loads were applied with a manually operated hydraulic ram and loads were measured with a load cell. Displacement instrumentation, was measured continuously during the test to failure. After the failure, the characteristics of all concrete breakout surfaces were documented.

Test bearing load levels were selected as $0.0f'_c$ (pure shear), $0.1f'_c$, $0.2f'_c$, $0.3f'_c$, and $0.4f'_c$. For most tests, the bearing load was held constant and the shear load was applied. For two tests, the bearing load was linearly increased simultaneously with the shear load. To assure full bearing on the plate, a sheet of greased Teflon® (PTFE) was placed on the bearing surface and a “wedding cake” of thick steel plates was stacked to gradually spread the bearing load. For all tests, the shear load was increased until failure occurred.
4 Test Results

Table 3 presents the test results, along with the actual measured values of the geometry and material properties. Due to concrete shear breakout surfaces being larger than anticipated, two test anchorages were damaged, with their breakout cones truncated. Observations of the failure surfaces follow:

**Low Bearing Stress** – These tests involve those with targeted bearing stress levels of 0.0$f'_c$ (pure shear), 0.1$f'_c$, and 0.2$f'_c$. The two, pure shear tests were typical shear breakout surfaces and served as calibration tests. For tests with target bearing levels of 0.1$f'_c$ and 0.2$f'_c$, the shear capacity developed was greater than the no bearing load condition. Figure 6 shows the breakout with a nominal bearing stress of 0.1$f'_c$. The concrete breakout surface was deeper and the entire shaft length (depth) of the stud was exposed; the entire stud length formed part of the final failure surface. (*Note in all the photographs, the stud head impression in the concrete was colored with a black marker, post-test, for reference.*) Instead of the top one or two diameters of the stud bearing on the concrete, it appeared the compressive confinement of the bearing load activated more stud length in resisting the shear force.

Figure 7 shows two test breakouts where the bearing stress was 0.2$f'_c$. Again the breakout was deeper and had a vertical back face. In addition, the breakout started to activate a second crack front propagating at the rear of the plate. This is illustrated better in Figure 8, looking down at the breakout. Again, the shear capacity of the anchorage exceeded both the test sets with 0.0$f'_c$ (pure shear) and 0.1$f'_c$ bearing levels.
**High Bearing Stress** — These tests involve targeted bearing stress levels of $0.3f'_c$ and $0.4f'_c$. The measured shear capacity of the anchorage exceeded that of the pure shear case, but the shear capacity increase measured in the $0.1f'_c$ and $0.2f'_c$ tests seemed to diminished. The shear strength appeared to peak between the $0.2f'_c$ and $0.3f'_c$ bearing load.

Figure 8 – Test BV22-24-6.4-1/2-7d-CC with a bearing stress of $0.2f'_c$.  
Figure 9 – Test BV24-24-6.4-1/2-7d-BCN with a bearing stress of $0.4f'_c$.  

Figure 9 illustrates the failure crack at ultimate load with a $0.4f'_c$ bearing stress. Note the crack propagation around the back edge of the plate. The higher bearing load activated a failure surface more representative of a bearing-type failure; the whole plate area was a contributor.

Figure 10 is a similar situation for a target $0.3f'_c$ bearing stress level. The failure crack propagated around the back edge of the plate (Figure 10(a)), and resulted in a fairly large shear breakout cone. Upon removal of the concrete piece in Figure 10(b), the failure surface revealed a sharp, vertical face corresponding to the rear crack face. The entire stud anchorage was buried in the breakout piece and the studs did not contribute to forming the breakout crack surface. The vertical face was approximately the depth of the headed studs. Edge bearing of the concrete seemed to be the dominant action.

Figure 10 – Test BV21-24-6.4-1/2-7d-BC with a bearing stress of $0.3f'_c$.  

(a) Breakout showing the development of the rear crack behind the anchor plate. The studs are contained in the breakout concrete.  
(b) Breakout concrete removed showing the vertical rear crack surface to a depth lower than the studs. The studs did not intersect the crack failure surface.
**Bearing Load Application** – Tests BV22-24-6.4-1/2-7d-AC and BV22-24-6.4-1/2-7d-CC are compared to BV22-24-6.4-1/2-7d-AL and BV22-24-6.4-1/2-7d-BL in this section. The former two tests used a constantly applied bearing load, whereas the latter two tests had the bearing load increasing during the test. No conclusions can be drawn from the two test sets as the failure loads were consistent. Although the goal was to increase the bearing load linearly, the load application became more of a step loading on account of the manual hand pump used for the ram.

5 **Data Analysis**

The shear load capacity is predicted given two methods. The first method is the PCI Design Handbook\(^2\) method formulated by the authors\(^3\). This method was derived from extensive experimental testing of headed stud anchorages. The second analysis method is the concrete capacity design (CCD) method from Chapter 17 of ACI 318-14\(^4\,5\).

Figure 11 shows the experimental behavior expressed as the test-to-predicted capacity versus the bearing stress referenced to the actual concrete compressive strength. The shear capacity was predicted with the average PCI shear equation using the PCI spacing factor for the two-stud pattern. The figure implies a beneficial effect of the bearing load on the connection up to \(f_{brg} / f_c'\) of about 0.42. For bearing stress levels, \(f_{brg} / f_c'\) of 0.1 to 0.3, the anchorage shear capacity increased and in some instances doubled (2x). As we observed during the testing, the peak effect seemed to occur around \(f_{brg} / f_c' = 0.2\). Above that level of bearing stress, the improved shear strength diminished as the bearing behavior governed at the higher bearing load(s).

The average concrete capacity design (CCD) shear prediction from Chapter 17 of ACI 318-14 is shown in Figure 12. The data trend of the plot is similar, but generally shifted upward for the headed studs. This demonstrates the CCD method is conservative when the edge distance is small (51 or 76 mm) and the spacing influence for the two-stud group is factored into the strength calculation for this close edge distance.

The influence of bearing on a shear-loaded anchorage is complicated, given the curvilinear nature of the relationship. A bearing load on the anchorage helps, but the benefits disappear when the bearing level is approximately 40 percent of the concrete compressive strength. We performed no tests above this 40 percent bearing level, so we recommend an upper limit on the influence. Given the nature of curtain wall or facade gravity loadings at an edge, this upper bound limit seems reasonable for this connection.

The failure surfaces at the higher bearing levels (\(0.3f_c'\) and \(0.4f_c'\)) were revealing with respect to behavior. At high bearing stress, the shear load seems to disrupt the edge bearing mechanism. At high bearing stress levels, the bearing mechanism is more dominant than the near-edge shear resistance. This behavior was evidenced by the rear boundary of the breakout surface being defined at the edge of the bearing plate. Moreover, the rear crack face was near-vertical before turning outward to the slab edge.
Figure 11 – Test-to-predicted (average) shear failure load (PCI) versus the bearing stress level expressed as a function of the concrete compressive strength.

\[ V_n = 15.7 \sqrt{f_c'} \left( c_{a1} \right)^{4/3} \left( 0.85 + \frac{s_1}{c_{a1}} \right) \]

Figure 12 – Test-to-predicted (average) shear failure load (ACI-CCD) versus the bearing stress level expressed as a function of the concrete compressive strength.

\[ V_n = \left( \frac{d}{d_a} \right)^{0.2} \sqrt{d_a} \sqrt{f_c'} c_1^{1.5} \left( \frac{A_v}{A_{v0}} \right) \]
The data presented a non-intuitive challenge to defining the bearing influence on a shear-loaded anchorage near an edge. The data can be fit to a curved, polynomial relationship, and the challenge was to simplify the bearing-shear interaction computation. We considered a tri-linear interaction curve, but viewed it to be too complicated. Additionally, we briefly studied the behavior of compressive-loaded shear lugs and there may be a correlation in the factors proposed in the literature.

Because the bearing influence on an anchorage will likely occur very near a front-edge, we selected the PCI Handbook equation as the basis for our analysis. Consequently, the selected modification factor for compressive bearing would be conservative for the CCD predictor. Given the two-stud anchorages of this test program, the best-fit relationship is shown in the plot of Figure 11; for the 13 tests, the correlation coefficient, R-squared, is 0.61. To provide a polynomial factor that passes through 1.0, a bearing effect modification factor for a shear-loaded anchorage is proposed as Equation 2:

$$\psi_{brg,v} = 17\eta(1 - 2.4\eta) + 1$$

where: $$\eta = \frac{f_{brg}}{f_c}$$ and $$0 \leq \eta \leq 0.42$$

The maximum bearing stress level is limited to 0.42 in this factor. This represents the present limitation of this research. In addition, bearing levels this high on a near-edge shear anchorage no longer appear to be an anchorage-to-concrete capacity determination; the behavior appears to be dominated by bearing. If the bearing stress is zero, the factor reduces to 1.0.

### 6 Conclusion

Additional research is recommended beyond these 11 multiple-anchor tests with combined bearing and shear to refine a bearing influence factor. The behavior was observed to change as the bearing stress was increased. At low bearing stress, less than about $$0.2f_c$$, the breakout crack propagated through the line of headed stud anchors. Above about $$0.3f_c$$, the breakout initiated at the rear of the plate and did not propagate through the stud anchors. The breakout crack also took on a new unique shape, propagating vertically to the depth of the anchor and then proceeding outward to the free edge of the concrete.

Bearing stress on an embed plate can have a beneficial effect but that benefit appears to be limited by a bearing stress near $$0.4f_c$$. Scatter exists in the 11 test samples reported. However, a reasonably simple bearing stress modifier is proposed for designing when combinations of bearing and shear exist simultaneously.

### 7 Acknowledgement

The authors wish to express their appreciation to the Wiss, Janney, Elstner Associates, Inc. (WJE) for the use of their testing equipment and portions of concrete specimens to cast these anchorages into a concrete slab. The authors would also like to thank the contributions of Messrs. Harry Chambers, Don Sues, and Don Mercker of Nelson Stud Welding for their donation of headed studs and stud welding services. Thanks also to Mr. Roger Becker, who at the time the testing was...
conducted was Vice President of Spancrete Industries, for Spancrete's casting of the test samples. The authors are also indebted the Precast/Prestressed Concrete Institute for initially sponsoring and extensive research project, which had a few leftover anchorage plates and headed studs because the test program scope changed as information on the shear behavior of welded headed stud anchors became available.

References


2. Precast / Prestressed Concrete Institute (2010), PCI Design Handbook, 7th Edition, (PCI MNL 120-10), Precast / Prestressed Concrete Institute, Chicago, Illinois, USA.


4. ACI Committee 318 (2014), Building Code Requirements for Structural Concrete (ACI 318-14), and Commentary on Building Code Requirements for Structural Concrete (ACI 318R-14) American Concrete Institute, Farmington Hills, MI, 524 pp.

Table 3 – Test results for the combined shear and bearing tests.

<table>
<thead>
<tr>
<th>Test Identification</th>
<th>Concrete Strength $f'c$ (MPa)</th>
<th>Test Plate</th>
<th>Test Geometry</th>
<th>Bearing Area (mm²)</th>
<th>Bearing Load B (kN)</th>
<th>Failure Load V (kN)</th>
<th>Bearing Stress, $f_{beg}$ (MPa)</th>
<th>PCI Test / Predicted</th>
<th>Failure Mode</th>
<th>PCI Test / Predicted</th>
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<tr>
<td>V23-2-7-BA</td>
<td>40.3</td>
<td>7A</td>
<td>51</td>
<td>89</td>
<td>15485</td>
<td>0.0</td>
<td>40.2</td>
<td>Conc.</td>
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<td>89</td>
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<td>15485</td>
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<td>Conc.</td>
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<td>15485</td>
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<td>Conc.</td>
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<td>1.95</td>
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Column (1): BV - (no. of studs) (bearing level: $n \times f'c$) – (bearing area) - ($h_d/d_a$ ratio) – (stud diameter) - ($s_2$ spacing) - (A, B, C, or D test)(C=constant bearing, L=increasing bearing, N=bearing non-concentric over studs) [units: inches]

BV = Bearing –Shear

Column (10): Bearing stress expressed as a percentage of the concrete compressive strength.

Column (12 & 13): Average equations used in the analysis.

Conversions: 1 in. = 25.4 mm, 1 ksi = 6.895 MPa, 1 lb = 4.448 N, 1 kip = 4.448 kN, 1 lb/ft³ = 16.03 kg/m³
In the framework of the present study concrete pryout failure mode, which occurs for relatively short and stocky cast-in or post installed anchorages, with embedment to diameter ratio $h_f/d < 4.5$ has been experimentally and numerically investigated. The behavior, failure mechanism and influencing parameters were investigated to clarify the dependency of the pryout capacity regarding the diameter of the fastener as well as anchor spacing perpendicular and parallel to the loading direction. The pryout capacity of both eccentrical shear and torsional loaded anchorages were investigated. Furthermore, the back and side edge influence as well as the corner influence on the pryout capacity were also studied. The results show that the pryout capacity increases proportional to the square root of the anchor diameter. Due to the application of pressure measurement film it was found that the compression stress distribution in front of the anchor shank takes place along the entire embedment depth. Furthermore, the increase of anchor spacing perpendicular to the shear load direction has not significant influence on the concrete pryout capacity, whereas the increase of anchor spacing parallel to the loading direction significantly increases the pryout capacity. For anchor spacing greater than a threshold spacing the pryout resistance of a group of anchors is equal to the resistance of a single anchor multiplied by the number of anchors. Currently available design codes are clearly conservative and mainly based on the experimental data base. Based on the results of the present study a mechanical model for the load-bearing behavior and failure mechanism of such anchorages is proposed. Moreover, the design formula to predict the concrete pryout capacity of single anchor is derived and extended through concrete capacity approach to account for an anchor group failing in pryout.

1 Introduction

Fasteners in concrete loaded in shear are frequently used in engineering practice for various connection types. The studies of the shear force transfer through the headed studs into the concrete slab goes back to the composite structures using push-off tests as reported by Viest\textsuperscript{1}, Driscoll and Slutter\textsuperscript{2}, Ollgaard et al.\textsuperscript{3} and others. In the push-off test setup a steel beam with welded headed stud anchors on it connects reinforced concrete slab and assure beam-slab shear load transfer. This kind of connection is typically used in bridge structures. On the other hand, in the case of pryout failure a single or multiple headed studs are welded on a single steel anchor plate, embedded in concrete and
Concrete pryout failure, which usually occurs in case of relatively shallow embedded anchors in concrete and loaded in shear, depends on several parameters such as concrete mechanical properties, embedment depth, stud diameter, anchor spacing and anchor plate size and thickness. Available provisions for predicting the pryout capacity are based mostly on statistical analysis of push-off test data. The pryout mechanism, which occurs away from all edge effects using short and stiff studs under direct shear load, was considered as the result of “kicking back” concrete behind the stud. The deformation of the stud shank results in compressive stresses in front of the shank and behind the stud head, which lever a breakout cone of concrete behind the stud. This bearing mechanism is analogous to a laterally loaded pile in earth hinged at the top, which is not taking into account the rotation of the anchor plate. On the other hand the adopted CEN/ACI model based on a pseudo-tension concrete breakout offers a suitable approach for predicting the pryout capacity. The model that is based on the mechanical transfer of shear load was recently discussed by Jebara et al. Pulling out the half-cone concrete breakout cone behind the anchor is based on the interaction between the tensile force in the stud and compressive force at the front side of the anchor. Based on the test results a modified mechanical model for single anchor is proposed. Moreover, pryout failure mode for a group of anchors with the influence of the edges and loading type is also investigated and discussed.

2 Single headed stud

The experimental investigation and a 3D finite element analysis of the performed tests on single anchor are presented and discussed. The pure shear tests without edge effects were conducted at the Institute for Construction Materials, University of Stuttgart (Germany). Three embedment depths of 30, 50 and 90 mm each with three stud diameters, ranging from 8 mm to 44 mm with embedment to diameter ratio $1.8 \leq h_e/d \leq 4.2$ were investigated. The anchor plate ($t_p$) and the stud head thickness ($h_k$) were for all anchorages larger than $0.5d$ and the stud-head-diameter to stud-diameter ratio $d_k/d=1.6$. The minimum yield and ultimate strength of the steel were 350 MPa and 540 MPa, respectively, for the anchorage plates and studs.

![Figure 1: Test setup and strong floor](image)

The anchorages were installed in four concrete slab specimens of 1.90 x 1.90 x 0.20 m in such a way to ensure enough space to establish concrete pryout failure. All anchorages were place at the top side of the formwork, so that the bottom plate surface flush with the concrete surface. The mean cube
concrete compressive strength \( (f_{cc}) \) at the day of testing was measured as 25.8 MPa. Note that the concrete quality was relatively low in order to assure concrete pryout failure. The slabs were reinforced with a minimum amount of reinforcement (mesh) for handling purposes.

The tests were carried out using strong floor for fixing the specimen including attached hole ram (Fig. 1). The loading surface of the pulling plate used for applying the shear load was 7 mm thick and was mounted on 1.5 mm thick Teflon stripe to avoid friction between steel plate and concrete. The load was monitored using a load cell. The horizontal and vertical deformations \( (\delta_h, \delta_v) \) were recorded with LVDT-s. The distributions of the internal bearing pressure, which develops under the anchor plate and at the front of the stud shaft, was measured using PRESCALE pressure measurement film (FUJIFILM-products). The PRESCALE Mono-Sheet type for high pressure measurement, which is 0.1 mm thick, was used.

![Figure 2: Typical finite element discretization and loading](image)

The finite element analysis of above mentioned anchors is carried out by the use of the 3D FE program MASA\textsuperscript{11,12}. The geometry and boundary conditions were adopted from the test setup. A typical finite element discretization of a single stud anchor loaded in shear is shown in Fig. 2. The modeled concrete slab thickness is \( h \geq 2h_{ef} \) with restrained bottom and side surfaces. The concrete is discretized with four node solid finite elements, whereas the welded headed stud and anchor plate are discretized using eight node solid elements (Fig. 2). The connection between steel and concrete is modeled using 1D contact elements (springs), which can take up only compressive forces. Note that the stiffness of the springs is taken as high as possible in order to simulate stiff connection of the concrete-steel interface in compression. The friction between anchor plate as well as the headed stud and concrete is not taken into account. The shear load is applied incrementally, perpendicular to the stud axis, in displacement control with 0.02 mm per load step. The material properties for concrete used in the analysis are: concrete uniaxial compressive strength \( f_{cc} = 21 \) [MPa], tensile strength \( f_{tc} = 1.7 \) [MPa], fracture energy \( G_F = 34 \) [J/m\(^2\)], Young’s modulus \( E_c = 27.5 \) [GPa] and Poisson ratio \( \nu_c = 0.18 \). The material properties used for steel are: Young’s modulus 200 [GPa] and Poisson ratio \( \nu = 0.33 \).

### 2.1 Influence of the stud diameter on the pryout capacity

The typical experimentally measured and numerically obtained load-displacement curves for the investigated embedment depths \( (h_{ef} = 30, 50 \) and 90 mm) with \( h_{ef} / d \approx 2 \) are plotted in Fig. 3a. It can be seen that the numerical results reasonably well replicate the experimentally measured load-displacement curves. The finite element computations yield to slightly stiffer response. This can be
explained by the fact that in such kind of problems the local non-linear effects (e.g. local crushing of concrete at the front of the stud) cannot be properly accounted for with the macroscopic type of analysis.

As mentioned above the stud diameter is one of the decisive influencing parameters on the pryout capacity. Fig. 3b shows the relative resistance $V_{cp} / V_{cp}^0$ versus stud diameter $d$, where $V_{cp}^0$ is the mean of shear resistance for the stud diameter $d = 16$ mm for the corresponding embedment depth. Thus, it is obvious that the concrete pryout capacity increases almost proportional to the square root of the stud diameter ($d^{0.5}$) for both, test and numerical results.

Increasing the stud diameter increases the activated concrete surface in front of the stud, which decreases the concrete compressive stresses and thereby increases the concrete resistance. This is quite clearly in case of short and stocky anchorages with $h_{ef}/d < 4.5$, which activates the concrete in front of anchor along the entire embedment depth.

2.2 Waterproof PRESCALE pressure measurement sheet

In order to provide an insight into the load-bearing behavior, in front of the stud shank and under the anchor plate a pressure measurement film was used. This film is applied for measuring pressure and pressure distribution between two surfaces and is widely applied in the industry, e.g. automobile, machinery and medical. An appropriate film is inserted between two surfaces to measure the pressure distribution by means of color density.

The PRESCALE Mono-Sheet type for high pressure measurement is used and is 0.1 mm thick. Color-developing and color-forming materials (microcapsules) are coated, one above the other, on a single base materiel (PET base). When the pressure is applied, the microcapsules are broken and the color-developing material absorbs the color-forming material and reacts generating a red color (Fig. 4a). Figure 4b illustrates waterproof PRESCALE sheet joined on the stud shaft and the anchor.
plate. To assure waterproof installation the Prescale sheets were joined at the stud shaft and the anchor plate using kapton (polyimid) tape, which is 0.07 mm thick and is also heat resistant.

![Diagram](image1)

**Figure 4**: (a) PRESCALE Pressure measurement film (FUJIFILM products guide), (b) Waterproof PRESCALE sheet and (c) PRESCALE pressure measurement film results

The measurement of pressure can be assessed by a color code. Areas where pressure is applied become red and the color density varies according to the intensity of the applied pressure (PRESCALE-PRODUCTS GUIDE, Fig. 4c).

It can be observed that the distribution of the compressive stresses takes place along the entire embedment depth and along a narrow strip under the front side of the anchor plate. Thus, the assumption of triangular compression stress in front of the stud shaft with maximum stresses near the concrete surface for stocky and stiff anchors, in the range of test anchorages, can be justified. The friction between anchor plate and concrete is concentrated along the narrow strip under the front side of the plate. The Prescale film, including the kapton-polyimid tape, assures frictionless connection of the stud shaft and the concrete.

Figure 5a shows numerically obtained typical distribution of the principal compressive stresses in concrete over the cross section plane parallel to the load direction. Dark zones show the highest principal compressive stresses in concrete in front of the stud along the stud shank. It can be seen that the distribution of compressive stresses is approximately linear over the anchor depth. Obviously, the numerical results confirm the results of above used Prescale pressure measurement films and the assumption of triangular stress distribution along the embedment depth. Consequently, the resulting shear force in front of the stud shank is assumed to be located at \( \frac{h_{ef}}{3} \) from the concrete surface.
Figure 5: FE-Analysis vs. test result for $h_{cf}/d \approx 2$ and $h_{cf} = 90$ mm: (a) compressive stresses, (b) principal strain and (c) typical cracking pattern

A typical damage zones (cracks) in terms of maximum principal strains and compressive stress at peak load for embedment depth $h_{cf} = 90$ mm with $h_{cf}/d \sim 2$ is shown in Fig. 5b. The red (dark) zone represents maximum principal strain that is larger than the critical crack opening strain $\varepsilon_{cr}$, which corresponds to the assumed critical crack opening $w_{cr} = 0.10$ mm ($\varepsilon_{cr} = w_{cr}/h$, $h$ = width of the crack band, i.e. effective element size according to the crack band method). The crack pattern obtained from the FE analysis reasonably matches the failure modes observed in the experiments. The localization of damage towards the concrete surface on the rear side of the anchorage and the concrete damage in front of the stud shank (max. principal strains) are in good agreement. The mean crack slope, which is the slope of the concrete breakout half-cone, obtained from the FE analyses agrees well with the concrete breakout slope observed in the tests ($\alpha_c = 30^\circ$). Note that for tensile pull-out failure the concrete breakout slope $\alpha$ is slightly larger ($\alpha = 35^\circ$). In case of pryout failure behind the anchor head there is a relative small zone of compressive stresses. These stresses together with tensile stresses caused by the pull-out force influence direction of crack, i.e. $\alpha_c$ decreases with increase of the mentioned compressive stresses.

2.3 Pryout failure mechanism - mechanical model

In order to improve Concrete-Capacity-Method to realistically account for the pryout capacity, it is important to clarify the transfer of the applied shear load into the concrete member. The principal load-transfer mechanism is shown in Fig. 6. The shear load $V$ is transferred basically through four components5: (1) Compressive force at the front of the headed stud or anchor bolt $D_{h,v}$; (2) Compressive force at the front side of the anchor plate $D_{p,v}$; (3) Tensile force in the headed stud or anchor bolt $N_{h,v}$ and (4) Friction between anchor plate and concrete $R_{p,v}$. The applied shear load $V$ develops bearing pressure in front of the stud, which results in internal overturning moment of the anchor plate. The ensuing rotation of the anchor plate induces bearing pressures in the concrete under the front side of the plate and tensile stresses in the stud. An increase in shear load increases the concrete stresses at front of the stud, which induces a zone of concrete crushing near the surface.
The concrete crushing near the surface is followed by increasing deformation of the stud and rotation of the plate. Hence the lever arm of the tensile force in the stud shank and the compression under the front side of the anchor plate produce the bending moment, which finally breaks out the concrete by pulling out the half concrete cone on the rear side of the anchorage. Therefore, the pryout mechanism could be treated as pseudo-tension concrete cone failure.
Based on these assumptions, a mechanical model was developed describing the pryout failure mechanism and thereby evaluating the ratio between tensile force in the stud and shear force at peak load. Fig. 7 illustrates the pryout failure mechanism as well as the mechanical model, which shows the interaction between the load (shear force) \( V \) and the reactions (resistance) of the concrete. Since the studs are typically very short and stiff, the bearing pressures in front of the stud could be assumed to have approximately a triangular stress distribution along the embedment depth, which was confirmed by the results of the experimental tests and numerical analysis. The concrete pryout capacity, which means pulling out the half concrete cone on the rear side of the anchorage, depends on the concrete resistance to the induced tension force \( N_{b,v} \) in the stud.

The tension force \( N_{b,v} \) was predicted from test data employing a simplified static system, which illustrates the deformed anchorage state at peak load (Fig. 7). The evaluation was done based on the following assumptions: (i) Anchor plate is rigid, (ii) Friction coefficient between front side of the anchor plate and concrete is \( \mu = 0.5 \), (iii) The average depth of the crushed concrete crater in front of the stud is approximately \( 0.5d \) and (iv) The compression stress in front of the stud shank has an approximately triangular stress distribution along the embedment depth. As can be seen from test results, the calculated tension force \( N_{b,v} \) seems to be approximately independent of anchor diameter (Fig. 8b). However, Fig. 8c,d show that the induced tensile force in the stud shank depends mainly on the applied shear load and the embedment depth. Thus, for specified shear load and corresponding embedment depth the tensile force in the stud depends only on the internal lever arm \( (l = a = 2d) \) and the applied shear load.

The above discussed load transfer mechanism is principally also valid for anchor groups and in cases when the anchor plate is flexible. However, for such cases the lever arm is not equal to \( 2d \) but depends on stiffness of the anchor plate and anchors. The same is the case for the steel-concrete composite beams where the so-called dowel action mechanism is activated. For all these cases the internal lever arm \( l \) should be obtained from the corresponding experiments. As shown in the evaluation of the test results (Fig. 8), the decrease in the stud stiffness leads to the decrease of the peak shear load and to an increase of the ratio between tensile force and shear resistance \( \eta_{N_v} = N_{b,v}/V_{cp} \). From the regression analysis of the tests results \( \eta_{N_v} \) is obtained as:
\[ N_{b,v}/V_{cp} = 0.34 \left( \frac{h_{ef}}{d} \right)^{0.39} \]  

### 2.4 Pryout capacity prediction of single anchor in-the-field

Based on the fact that the tension force directly depends on the applied shear load, it is possible to derive a relation for shear loaded anchorages based on the concrete breakout pyramid model. To predict the pryout capacity the tension force in the stud, calculated according to the proposed mechanical model, can be related to the simplified half-pyramid concrete breakout model. By means of the tension force and the equivalent projection area, the effective ultimate concrete stress \( \sigma_{b,v} \) for shear loaded anchorages is calculated as:

\[ \sigma_{b,v} = \frac{N_{b,v}}{A_{p,v}} \]  

In the idealized pyramid concrete breakout model (see Fig. 9a) the equivalent projection area \( A_{p,v} \), which depends on the decline of the breakout half-cone surface \( \alpha_{v} \) and the embedment depth \( h_{ef} \), is calculated as:

\[ A_{p,v} = 2 \cdot S_{k,v}^2 = 2 \cdot \left( \frac{h_{ef}}{\tan \alpha_{v}} \right)^2 \]  

where \( S_{k,v} \) is a half size of the pyramid base. This rather simplified assumption follows approach of current CC design method\(^4\) and was also previously verified\(^5,6\).

Since the induced tension force \( N_{b,v} \) in the stud depends on the shear load \( N_{b,v} = f(V_{cp}) \) one can write:

\[ f(V_{cp}) = \sigma_{b,v} \cdot A_{p,v} \]  

Evaluating the tension force to pryout capacity ratio, \( \eta_{Nv} = N_{b,v}/V_{cp} \), from experiments and inserting it into (1) yields to the equation for the pryout capacity of single stud:

\[ V_{cp} = \sigma_{b,v} \cdot A_{p,v} / \eta_{Nv} \]  

The experimental results\(^7\) show that the mean value of the evaluated effective ultimate concrete stress \( \sigma_{b,v} \) (performance ratio for the concrete tensile strength) is nearly twice as much as the effective ultimate concrete stress for direct tensile concrete cone failure \( (\sigma_{b,T}) \), \( \sigma_{b,v} = 2 \cdot \sigma_{b,T} = 0.3 \cdot f_{cc}^{0.5} \) (see Fig. 2.9b), where \( f_{cc} \) is the compressive strength of concrete cube. Note that according to direct tensile pull-out tests the effective ultimate concrete stress relating to the projection area of the concrete breakout pyramid is approximately \( \sigma_{b,T} = 0.15 \cdot f_{cc}^{0.5} \) [5,15]. The doubling of the effective ultimate tensile concrete stress for shear tests is attributed to the positive effect of compressive force of the anchor plate, which increases the concrete tensile resistance.

The relatively high scatter of the effective ultimate concrete stress is related to the scatter of the half-cone surface slope. Namely, higher concrete half-cone surface slope yields to small projected area which results in high effective ultimate concrete stress. Nevertheless, it is possible to verify this assumption analytically. Following the assumptions of Zhao\(^5\) it is possible to integrate the tensile stresses over the half-cone surface at peak load and to calculate the resulting ultimate tensile force generated in the stud. Dividing this ultimate force by the assumed projected half-pyramid base surface \( A_{p,v} \) yields to the effective ultimate tensile resistance of approximately \( 0.30 \cdot (f_{cc})^{0.5} \).
By inserting the result of the test data regression analysis for $\sigma_{b,v}$ and $\eta_{Nv}$ together with the equivalent projection area $A_{p,v}$ into (5), the following rounded mean value equation for concrete pryout capacity is obtained:

$$V_{cp} = 6 \cdot d^{0.5} f_{cc}^{0.5} \cdot h_{ef}^{1.5}$$

(6)

The current provision of the ACI-318 Appendix D and CEN/TS for pryout capacity of a single anchor is a two-stage step function. It depends on the embedment depth and is based on the tensile concrete breakout model:

$$V_{cp} = k_3 \cdot N_{u,c}$$

(7)

where $k_3 = 1$ for $h_{ef} < 60$ mm, $k_3 = 2$ for $h_{ef} \geq 60$ mm and $N_{u,c}$ is the basic average equation for concrete tensile breakout strength of a single cast-in-stud anchor in un-cracked concrete given by:

$$N_{u,c} = 15.5 f_{cc}^{0.5} \cdot h_{ef}^{1.5}$$

(8)

It should be noted that, in contrary to (6), (7) does not depend on the diameter of the anchor bolt. The comparison of the pryout capacity of the tests using CEN/TS, ACI 318 Appendix D (7, 8) results in the prediction mean of 1.31, standard deviation 0.33 and COV of 25.1% (see Fig. 10). On the other hand, comparing the above test results and available test results with the prediction pryout capacity using (6) yields to the prediction mean of 1.03, the standard deviation of 0.11 and the coefficient of variation (COV) of 11% (see Fig. 10).

As can be seen from Fig. 10 the CEN/TS, ACI 318 Appendix D predicted pryout capacities are clearly conservative and underestimate the true pryout capacity of anchorages with low embedment depth to diameter ratios. On the other hand, the proposed model yields to realistic prediction and more effectively utilizes the full capacity of the short, stocky anchors under shear loads.
3 Conclusions

In the present study the behaviour of short welded anchor bolts installed in normal strength concrete and loaded in shear was investigated. The tests carried out are aimed at clarifying the pryout mechanism with particular emphasis on the influence of the stud diameter on the pryout capacity. The stud diameter was varied keeping the embedment depth to diameter ratio $h_{ef}/d \leq 4.2$, in order to invoke the pryout failure mode. The PRESCALE sheet films were used to provide an insight into the distribution of compressive stresses in front of the anchor and under the steel plate. Based on the results of the study the following can be concluded. (1) The pryout mechanism for welded stud anchorages can be considered as a pseudo-tension (indirect tension) breakout. The force pair, stud in tension and plate in compression, induce the concrete breakout on the back side of the anchorage. (2) A triangle compression stress distribution in front of the stud shaft can be adopted for the mechanical model. (3) The pryout capacity of the stud anchorages embedded in normal weight concrete is influenced by the stud diameter and is approximately proportional to its square root ($d^{0.5}$). (4) The proposed formula for pryout capacity of single welded stud in normal weight concrete based on the pseudo-tension model can be used to improve current design formulas. This is especially true for stiff anchors for which the pryout failure mode can be critical. (5) The formula is valid under the assumptions that both the anchor plate and anchor are relatively stiff, anchor plate overhang is approximately $2d$ and the head to stud diameter ratio should be in the range close to present experimental investigations ($d_k/d=1.6$). Most of headed stud anchorages in engineering practice meet these conditions. (6) The employed finite element code that is based on the microplane material model for concrete can realistically simulate the behaviour of investigated fastener for the pryout failure mode. The numerical results agree well with the results of recently performed tests.

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4. Hawkins, N., “Strength in Shear and Tension of Cast-in-Place Anchor Bolts”, Anchorage to Concrete, SP-103, American Concrete Institute, Detroit, MI, 235-255, 1987


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BEHAVIOUR OF MONO-STUD PLATES IN CRACKED CONCRETE UNDER SHEAR LOADING

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ABSTRACT

Anchor plates are commonly used in nuclear power plants as connection between equipment and reinforced concrete structures. These anchoring solutions are usually cast-in place, thus the steel plate is embedded in the concrete. The contribution of the embedded plate is usually neglected in calculation codes however, in this study, it is demonstrated that its contribution is noticeable. Classic anchor plates typically have four fasteners, thus complicating the installation with high density of reinforcement. New specific solutions with only one headed anchor are investigated in this paper: a more classic solution made with a single stud welded in the centre of the steel plate, and a more innovative one made with a piece of an IPE profile, welded in the centre of the steel plate. An experimental campaign was carried out in order to investigate the shear behaviour in cracked concrete with different boundary conditions. Confined and non-confined conditions were applied in order to manage the failure mode (steel failure of the stud or concrete edge failure) and in order to control the border effect of the embedded plate (with or without contact between concrete and edge plate). The contributions to the shear capacity of both stud, concrete and plate are evaluated. Finally, experimental results are compared with Concrete Capacity Design (CCD) method.

1 Introduction

Safety of strategic constructions is related to the capacity of both structural and non-structural elements. Non-structural parts are usually connected to structures via fastening. Anchoring plates are a common solution to create a link between equipment and reinforced concrete structures in nuclear power plants. Despite the wide diffusion of post-installed fasteners in recent years, many constructive details require cast-in place anchors. The main distinction among post-installed and cast-in anchors is function of the installation procedure: after concrete curing the former, and during concrete casting the latter\textsuperscript{1}. In nuclear power plants, using of cast-in anchoring solutions is often preferred in order to avoid additional sealing operations

Anchoring plates are composed by headed anchors welded to the steel plate. Typical solutions use four studs, thus complicating the installation in regions where high reinforcement ratio is expected. Electricité de France (EDF) developed innovative solutions for anchor plates to be used in these design conditions under seismic loads.
Two experimental campaigns were carried out by Department of Civil and Environmental Engineering of Politecnico di Milano (DICA) for the assessment of these anchoring solutions: the former covered the tensile behaviour, while the latter treated the shear behaviour. Due to the lack of specific protocols for the assessment of cast-in-place anchors, existing protocols initially developed for post-installed anchors were adapted.

In this paper, monotonic shear tests of mono-stud anchor plates with different boundary conditions in cracked concrete are presented. Tests were carried out at Laboratorio Prove e Materiali, Strutture e Costruzioni (LPMSC) of Politecnico di Milano. In order to manage different failure mechanisms, confined and non-confined conditions were applied. Specifically, steel failure and concrete edge breakout were achieved in tests with and without confinement, respectively. None of the tested specimens failed with concrete pry-out mechanism. Five combinations of load and boundary conditions were accounted: confined tests under centred and eccentric shear, confined tests under eccentric shear without embedding of the plate, unconfined tests under centred shear with or without contact between steel plate and concrete.

Results of tests are discussed to evaluate contributions of each component (stud, plate and concrete). Comparison between results and theoretical predictions of CCD method are reported to prove its applicability as design method for these innovative solutions.

2 Experimental investigation

2.1 Mono-stud anchor plates

Two anchoring plates were tested:

- Solution S1 (Figure 1): a more classic solution made with a stud of 22mm diameter and 200mm height welded in the centre;
- Solution S4 (Figure 1): a more innovative solution made with a piece of IPE profile of steel S235 of 73mm length welded in the centre.

Class S355 steel plates are intended to accommodate a welded support for equipment with a maximum distance of 50mm with respect to the centroid axis. Table 1 summarizes geometrical details of mono-stud plates. Two other solutions, named (S2 and S3) were also developed but they are not addressed by this paper.

According to FprEN 1992-4, the embedment depth is 212mm and 155mm for solution S1 and solution S4, respectively.
Table 1: Geometry of mono-stud anchor plates.

<table>
<thead>
<tr>
<th>Solution</th>
<th>Geometric Details</th>
<th>Plate (mm)</th>
<th>studs</th>
<th>I beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td></td>
<td>200×200×22</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>S4</td>
<td></td>
<td>200×200×22</td>
<td>0</td>
<td>1</td>
</tr>
</tbody>
</table>

2.2 Concrete slabs

In order to ensure the correct development of failure mechanisms, two different slabs were designed. A reinforced concrete slab of 500mm width, 340mm depth and 1440mm length with two anchoring plates were selected for confined shear tests (Figure 2). Slabs of same dimensions were used for unconfined shear tests but with only one specimen for each slab (Figure 2). The installation of one specimen for each slab is required for the complete development of concrete edge breakout.

![Figure 2: Geometry of slabs: confined tests (upper), unconfined tests (lower).](image)

Slabs were casted with class C20/25 concrete and reinforced with M20 steel class B7 threaded rods. Contact between the steel plate and the concrete was removed by applying shaped pieces of polystyrene before concrete casting (Figure 3). For each slab, at least three compressive test on cubes of 150mm side were performed to measure the average compressive strength of the concrete.

![Figure 3: Steel plates: full contact (left), lateral contact (middle), no contact (right).](image)
As shown in Figure 2, crack inducers ensure the localization of crossing cracks to studs.

### 2.3 Protocols adopted

Nowadays there is a lack of information about the assessment of cast-in place anchors. On the other hand, post-installed anchors are qualified according to specific procedures, which are provided by ETAG001 Annex E and ACI 355.2 in Europe and United States, respectively. Additionally, DIBt published a national guideline for post-installed anchors to be used in nuclear power plants.

Other authors collected data from several research works in order to review tests for the evaluation of shear strength for the seismic behaviour of headed studs. A complete review of test protocols for the assessment of anchors under seismic loading is beyond the scope of this work. Tests which are presented in the present paper belong to a more extensive experimental campaign. The complete experimental campaign is composed by the following tests:

- Monotonic reference tension test;
- Pulsating tension test with a fixed value of crack width;
- Monotonic reference shear test;
- Alternating shear test with a fixed value of crack width;
- Tension load with varying crack width.

Protocols of ETAG001 Annex E were adapted for the scope of this work. Specifically, tests for category C2 were selected, hence monotonic shear tests are identified by code the C2.2 hereinafter. Tests were carried out in cracked concrete with fixed crack width of 0.8mm. Although this value is smaller with respect to crack width prescribed by DIBt code for Ultimate Limit State condition (ULS), it is assumed that 0.8mm is sufficiently representative of ULS for non-structural components. Only for Solution S1 and for confined tests under eccentric shear without embedding of the plate, a specific protocol composed of three phases was applied:

- First monotonic shear loading up to 20mm of displacement;
- Alternating shear loading with 35 cycles of ±8mm of displacement;
- Second monotonic shear loading up to 20mm of displacement.

The main goal of this protocol is to investigate the degradation of adhesion between concrete and lateral surface of stud.

### 2.4 Load and boundary conditions

In order to achieve cracked conditions in concrete, the shear was always applied parallel to the crack. According to Eligehausen, if the shear load acts perpendicular to the crack, the load-displacement behaviour does not differ significantly from the behaviour in non-cracked concrete. Additionally, direction of shear load with respect to the crack is even more important for Solution S4 in which the body of the anchor is not axisymmetric. Indeed, for Solution S4 two conditions are possible: the former with the crack perpendicular to the web of IPE profile (the anchor behaves like in non-cracked concrete) and the latter with the crack parallel to the web of IPE profile (the anchor behaves like in cracked concrete). Accordingly, the IPE profile was always oriented with the web parallel to the crack.
According to EDF\textsuperscript{2}, anchoring plates are designed to allow the installation of supports for equipment with a maximum eccentricity of 50mm.

Consequently, two limit cases are identified:

- Centred shear with the load direction coincident with the axis of the anchor;
- Eccentric shear with 50mm of eccentricity with respect to the axis of the anchor.

These load conditions are coupled with enforced boundary conditions for the slab in order to evaluate the contribution to the shear capacity of each component (Figure 4). Hence, specific designation of tests is introduced in Table 2.

<table>
<thead>
<tr>
<th>Code</th>
<th>Shear load</th>
<th>Edge of the slab</th>
<th>Expected failure mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2.2-C</td>
<td>Centred</td>
<td>Bounded</td>
<td>Steel failure</td>
</tr>
<tr>
<td>C2.2-E</td>
<td>Eccentric</td>
<td>Bounded</td>
<td>Steel failure</td>
</tr>
<tr>
<td>C2.2-E-T</td>
<td>Eccentric</td>
<td>Bounded</td>
<td>-</td>
</tr>
<tr>
<td>C2.2-C-U</td>
<td>Centred</td>
<td>Free</td>
<td>Concrete edge breakout</td>
</tr>
<tr>
<td>C2.2-C-UC</td>
<td>Centred</td>
<td>Free</td>
<td>Concrete edge breakout</td>
</tr>
</tbody>
</table>

Figure 4: Boundary conditions: a) confined test without contact, b) confined test without embedding of the plate, c) unconfined test without contact, d) confined test with contact.

### 2.5 Test setup and execution

A special apparatus composed of two reaction frames was used for tests (Figure 5). The first frame lays horizontally and it is fixed to the strong floor. It is equipped with a hydraulic jack with a maximum load capacity of 1000kN. Tensile load is applied to the slab to open cracks in the concrete slab up to the value fixed by protocols (0.8mm). The second frame is movable and it was positioned...
on the concrete slab. It is composed by three portions bolted each other: a horizontal hydraulic jack of 300kN capacity, a reactive frame on which the jack is bolted, coupled steel profiles with a sliding plate over rails which connects anchoring plate to the jack.

Unconfined conditions were enforced by slightly modifying the clamping system of the movable horizontal frame. A clamping system composed by two steel plates and a portal frame with a clear space of 1000mm length and 25mm depth were used for confined tests and unconfined tests, respectively (Figure 6).

Linear displacement transducers (LVDTs) of different length were installed on concrete slabs to measure both crack width and displacements of anchor plates. For eccentric shear tests, displacements of anchor plates were measured at both side of the plate.

Totally, thirty-eight tests were carried out. The complete test program is reported in Table 3.
Table 3: Test program.

<table>
<thead>
<tr>
<th>Test code</th>
<th>Solution</th>
<th>Concrete</th>
<th>Crack width (mm)</th>
<th>Eccentricity (mm)</th>
<th>N. of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-C2.2-C</td>
<td>S1</td>
<td>C20/25</td>
<td>0.8</td>
<td>-</td>
<td>5</td>
</tr>
<tr>
<td>S1-C2.2-E</td>
<td>S1</td>
<td>C20/25</td>
<td>0.8</td>
<td>50</td>
<td>5</td>
</tr>
<tr>
<td>S1-C2.2-E-T</td>
<td>S1</td>
<td>C20/25</td>
<td>0.8</td>
<td>50</td>
<td>2</td>
</tr>
<tr>
<td>S1-C2.2-C-U</td>
<td>S1</td>
<td>C20/25</td>
<td>0.8</td>
<td>-</td>
<td>5</td>
</tr>
<tr>
<td>S1-C2.2-C-UC</td>
<td>S1</td>
<td>C20/25</td>
<td>0.8</td>
<td>-</td>
<td>5</td>
</tr>
<tr>
<td>S4-C2.2-C</td>
<td>S4</td>
<td>C20/25</td>
<td>0.8</td>
<td>-</td>
<td>5</td>
</tr>
<tr>
<td>S4-C2.2-E</td>
<td>S4</td>
<td>C20/25</td>
<td>0.8</td>
<td>50</td>
<td>5</td>
</tr>
<tr>
<td>S4-C2.2-E-T</td>
<td>S4</td>
<td>C20/25</td>
<td>0.8</td>
<td>50</td>
<td>2</td>
</tr>
<tr>
<td>S4-C2.2-C-U</td>
<td>S4</td>
<td>C20/25</td>
<td>0.8</td>
<td>-</td>
<td>2</td>
</tr>
<tr>
<td>S4-C2.2-C-UC</td>
<td>S4</td>
<td>C20/25</td>
<td>0.8</td>
<td>-</td>
<td>2</td>
</tr>
</tbody>
</table>

3 Test results

Table 4 summarizes results of experimental tests, where $f_{c,\text{test}}$ is the compressive strength of the concrete batch, letter S means steel failure, letter C means concrete edge breakout failure, $\delta_{0.5V_{u,m}}$ is the displacement at the 50% of the peak load, $V_{u,m,\text{test}}$ and $V_{u,m}$ are the average shear load and the average shear load normalized, respectively. It worth be remarked that normalization of the ultimate shear load is calculated only when concrete edge breakout failure is achieved using the following equation:

$$V_{u,m}(f_c) = V_{u,m,\text{test}} \cdot \left(\frac{f_c}{f_{c,\text{test}}}\right)^{0.5}$$

Table 4: Results of experimental tests

<table>
<thead>
<tr>
<th>Test series</th>
<th>Solution</th>
<th>Number of tests</th>
<th>$f_{c,\text{test}}$ (N/mm²)</th>
<th>Type of failure</th>
<th>$\delta_{0.5V_{u,m}}$ (mm)</th>
<th>CoV($\delta_{0.5V_{u,m}}$) (%)</th>
<th>$V_{u,m,\text{test}}$ (kN)</th>
<th>$V_{u,m}$ (kN)</th>
<th>$\sigma$ ($V_{u,m}$) (kN)</th>
<th>CoV($V_{u,m}$) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2.2-C</td>
<td>S1</td>
<td>5</td>
<td>18.0/32.4</td>
<td>S</td>
<td>6.87</td>
<td>33.3</td>
<td>182.0</td>
<td>-</td>
<td>17.3</td>
<td>9.5</td>
</tr>
<tr>
<td>C2.2-E</td>
<td>S1</td>
<td>5</td>
<td>18.0/32.4</td>
<td>S</td>
<td>6.69</td>
<td>20.3</td>
<td>177.9</td>
<td>-</td>
<td>7.4</td>
<td>4.1</td>
</tr>
<tr>
<td>C2.2-E-T</td>
<td>S1</td>
<td>2</td>
<td>37.1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>13.3</td>
<td>10.9</td>
<td>1.4</td>
<td>10.6</td>
</tr>
<tr>
<td>C2.2-C-U</td>
<td>S1</td>
<td>5</td>
<td>27.1/28.9</td>
<td>C</td>
<td>4.01</td>
<td>29.8</td>
<td>92.8</td>
<td>88.0</td>
<td>12.4</td>
<td>13.4</td>
</tr>
<tr>
<td>C2.2-C-UC</td>
<td>S1</td>
<td>5</td>
<td>27.1/34.4</td>
<td>C</td>
<td>2.47</td>
<td>19.6</td>
<td>97.2</td>
<td>84.9</td>
<td>5.9</td>
<td>6.1</td>
</tr>
<tr>
<td>C2.2-C</td>
<td>S4</td>
<td>5</td>
<td>18.0/29.0</td>
<td>S</td>
<td>3.93</td>
<td>24.1</td>
<td>158.4</td>
<td>-</td>
<td>9.9</td>
<td>6.3</td>
</tr>
<tr>
<td>C2.2-E</td>
<td>S4</td>
<td>5</td>
<td>29.0/29.2</td>
<td>S</td>
<td>5.16</td>
<td>12.1</td>
<td>178.4</td>
<td>-</td>
<td>15.4</td>
<td>8.6</td>
</tr>
<tr>
<td>C2.2-E-T</td>
<td>S4</td>
<td>2</td>
<td>25.6</td>
<td>S</td>
<td>5.40</td>
<td>0.1</td>
<td>126.5</td>
<td>-</td>
<td>4.8</td>
<td>3.8</td>
</tr>
<tr>
<td>C2.2-C-U</td>
<td>S4</td>
<td>2</td>
<td>25.6</td>
<td>C</td>
<td>2.16</td>
<td>24.9</td>
<td>97.0</td>
<td>95.8</td>
<td>0.8</td>
<td>0.9</td>
</tr>
<tr>
<td>C2.2-C-UC</td>
<td>S4</td>
<td>2</td>
<td>25.4/27.4</td>
<td>C</td>
<td>2.39</td>
<td>5.3</td>
<td>93.9</td>
<td>91.3</td>
<td>10.0</td>
<td>10.6</td>
</tr>
</tbody>
</table>
4 Discussion of results

4.1 Confined tests

Solution S1 shows same results both under centred and under eccentric shear loading (Figure 11). Comparison of failure surfaces reveals no difference between centred and eccentric tests (Figure 7).

Due to the small torsional resistance of the headed stud, only the contribution of the concrete in contact assures torsional resistance. The capacity of the anchoring plate is negligible when this contribution is removed. Test series C2.2-E-T supports this evidence: when Solution S1 is loaded under eccentric shear without embedding of steel plate, it rotates with only a small resistance generated by friction between concrete and lateral surface of the stud (Figure 8). Furthermore, the frictional resistance is completely lost after cyclic loading (Figure 8).

Capacity of Solution S4 under eccentric shear is comparable with capacity under centred shear (Figure 11) due to the development of contact forces between the concrete and the web of IPE profile. Failure surfaces in Figure 9 clearly show torsional deformation of the web of IPE profile under eccentric shear. This evidence is confirmed in test series C2.2-E-T, where a significant capacity was detected despite the absence of contact between the steel plate and the concrete (Figure 10). Additionally, it is found that the contact of the embedded plate (test series C2.2-E) increases the ultimate capacity of 40%, but it does not affect the initial stiffness (Figure 10).
Experimental results are compared with theoretical values of CCD method\textsuperscript{1,4} (Figure 11). The load capacity in shear is expressed by the following equations with reference to mean and not characteristic values:

\begin{equation}
V_{\mu,m,S1} = \alpha_1 \cdot A_s \cdot f_u
\end{equation}

\begin{equation}
V_{\mu,m,S4} = \alpha_1 \cdot A_s \cdot f_u + \beta_c \cdot b \cdot t_f \cdot f_c
\end{equation}

where $\alpha_1$ is taken as equal to 0.6 and 0.7 for Solutions 4 and 1, respectively, to take into account the increase in the resistance due to the welded plate\textsuperscript{1,10}, $\beta_c \cdot b \cdot t_f \cdot f_c$ is the contribution of the top flange of IPE profile in contact with concrete. $\beta_c$ is equal to 1.0 or 0.25 for centred and eccentric shear, respectively.
4.2 Unconfined tests

Ultimate shear loads for of C2.2-C-U and C2.2-C-UC tests series are comparable. Indeed, cracks from edge in contact with concrete appear before the ultimate load. Crack patterns of test series are depicted in Figure 12 and Figure 13, in which continuous lines and dashed lines highlight concrete edge breakout of anchor and border breakout of embedded plate, respectively.

![Figure 12: Solution S1: crack pattern of C2.2-C-U test (left) and C2.2-C-UC test (right).](image)

![Figure 13: Solution S4: crack pattern of C2.2-C-U test (left) and C2.2-C-UC test (right).](image)

Contact of the embedded plate with concrete does not increase the ultimate shear load. On the other hand, assuming the stiffness at 50% of ultimate shear load as representative of the behavior at Serviceability Limit State (SLS)\(^210\), it strongly affects the stiffness for Solution S1; conversely, there is no increasing of stiffness for Solution S4 (Figure 14).

![Figure 14: Load-displacement curve for confined tests: Solution S1(left) and Solution S4 (right).](image)

Experimental results are compared with theoretical predictions provided by CCD method\(^14\) (Table 5 and Figure 15, where \(V_{ct,m,test}\) and \(V_{ct,p,test}\) are the average normalized contact load and the predicted contact load, respectively, \(k_{0.5V_{u,m,U}}\) and \(k_{0.5V_{u,m,UC}}\) are the average stiffness at the 50% of the peak...
load for unconfined test without and with contact, respectively. As in the previous section, the load capacity in shear is expressed with reference to its mean value:

\[ V_{Rm,c}^0 = 3.0 \cdot d_{nom}^2 \cdot \eta^3 \cdot \beta_t \cdot \gamma_{et}^{0.5} \cdot c_1^{1.5} \]  

(4)

The same equation is adopted for both the prediction of concrete edge breakout and border effect of the embedded plate.

Table 5: Comparison of results of unconfined tests.

<table>
<thead>
<tr>
<th>Solution</th>
<th>( V_{ct,m,test} ) (kN)</th>
<th>( \sigma (V_{ct,m,test}) ) (kN)</th>
<th>( \text{CoV} (V_{ct,m,test}) ) (%)</th>
<th>( V_{ct,p,test} ) (kN)</th>
<th>( V_{ct,m,test} / V_{ct,p,test} ) (-)</th>
<th>( k_{S5,U} ) (kN/mm)</th>
<th>( k_{S5,U,UC} / k_{S5,U} ) (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>53.4</td>
<td>8.0</td>
<td>13.1</td>
<td>51.4</td>
<td>1.04</td>
<td>10.30</td>
<td>20.47</td>
</tr>
<tr>
<td>S4</td>
<td>61.8</td>
<td>0.4</td>
<td>0.6</td>
<td>51.4</td>
<td>1.20</td>
<td>20.62</td>
<td>19.61</td>
</tr>
</tbody>
</table>

Figure 15: Comparison between tested and predicted capacity of unconfined tests for both solutions.

CCD method slightly overestimates capacity of Solution S1. The difference between experimental and theoretical values is probably due a different angle of the concrete breakout with respect to CCD method. Indeed, angles in a range of 35°-45 degrees were detected. Results of Solution S4 are comparable with prediction of CCD, however prediction of border effect is lower than experimental values. Probably a different failure mechanism is activated involving the top flange of IPE profile, which has a non-negligible dimension.

5 Conclusions

The following conclusions can be drawn:

- Solution S1 has negligible resistance under eccentric shear in confined condition, if the steel plate is not embedded in concrete;
- Solution S4 has noticeable capacity also when the plate is not embedded in concrete, both in confined and unconfined conditions;
- Embedded steel plate has no influence on the ultimate shear load for both anchoring plates in concrete edge breakout;
- Embedded steel plate has strong influence on the stiffness of Solution S1 but it does not affect the stiffness of Solution S4;
It is strongly suggested to take care of the compaction of concrete around the plate for Solution S1;
- CCD method can be used as design method for these innovative solutions but minor calibrations of coefficients are needed.

6 Acknowledgement

This research work was supported by EDF SEPTEN Company, Civil Engineering, Villeurbanne. Mr. A. El Yazidi and Mr. T. Roure at EDF are warmly thanked. The authors also thank all the members of technical staff if LPMSC, sector Structural Anchors, with a special mention to Mr. M. Dezio.

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COMPREHENSIVE EXPERIMENTAL INVESTIGATIONS ON ANCHORAGES WITH SUPPLEMENTARY REINFORCEMENT

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ABSTRACT

The behavior of anchorages with multiple headed studs is significantly influenced by the presence of supplementary reinforcement. The supplementary reinforcement to resist tension forces on the anchorage consists of surface reinforcement and the stirrups, usually enclosing the surface reinforcement. For the anchorages placed close to the edge and loaded in shear towards the edge, the supplementary reinforcement in the form of edge reinforcement and the stirrups enclosing edge reinforcement can be provided. The behavior of anchorages with supplementary reinforcement can be best described using a strut-and-tie model in which the forces applied to the anchorage are resisted by a network of concrete struts taking up the compression forces and tension ties formed by the surface (or edge) reinforcement and the stirrups. Thus, in principle, there are three major components in this strut-and-tie model, namely the concrete struts, the tension ties and the nodes. Increasing the amount of supplementary reinforcement leads to an increase in the failure load of the tension ties. However, beyond a certain level of reinforcement, the failure of concrete struts can limit the failure load for the anchorage.

In this work, detailed experimental investigations were carried out on anchorages (WELDA® anchor plates from Peikko Group Corporation) consisting of multiple headed studs with supplementary reinforcement to take up tension forces subjected to tension loads and anchorages with supplementary reinforcement to take up shear forces close to an edge loaded in shear perpendicular and towards the edge. The experimental program was designed to capture the behavior of the different components and the forces taken up by concrete and reinforcement were segregated using the data obtained from strain gauges applied on the stirrups. It was clearly brought out that several assumptions made in the existing models e.g. the models of EN1992-41, ACI 3182 or fib Bulletin 583 are not entirely in accordance with the real behavior and therefore the models are either overly conservative or tend to become unconservative depending on the configuration and the amount of supplementary reinforcement. Based on the detailed evaluation of the experimental results, a new model is developed that can rationally consider and capture the realistic behavior of the anchorages with supplementary reinforcement. The model is presented in accompanying paper.
1 Introduction

The anchor (supplementary) reinforcement has a significant influence on the load-carrying capacity of anchorages subjected to tension or shear loads. The load-bearing capacity of an anchorage with supplementary reinforcement results from the anchorage in the concrete breakout body achieved by means of bond and bearing of a hook or bend. When the anchorage is loaded in tension or shear close to and towards the edge, first the concrete cracks forming the breakout body, and then the stirrups get activated. If the anchorage length of the stirrups within the concrete breakout body is small, they might reach bond failure prior to yielding resulting in lower resistance than potentially achievable. For stirrups with relatively large anchorage lengths, as in case of closely spaced stirrups, resistance equal to the yield resistance of the stirrup can develop resulting in enhanced load and deformation capacity of the anchorage. However, beyond a certain level of reinforcement, the failure of concrete struts can limit the failure load for the anchorage.

In this work, systematic investigations were made on quadruple anchorages (2x2 anchor groups), in the form of WELDA® anchor plates from Peikko Group Corporation, without and with supplementary reinforcement under tension loads and under shear loads close and towards the edge. Different amounts of supplementary reinforcement were used in the tests targeting the reinforcement and concrete failure modes. This paper discusses the details of the experimental program as well as detailed evaluation of the test results. An analytical model developed based on these test results is presented in accompanying paper.

2 Experimental investigations

2.1 Anchorages

The quadruple (2x2) anchor groups (WELDA® anchor plates from Peikko Group Corporation) were tested in this program. Figure 1 presents the geometrical details of the anchor groups tested in this program. The effective depth of the anchors was $h_{ef} = 155$ mm.

Figure 1: Anchorage configuration used for tension and shear tests
2.2 Supplementary reinforcement

For tension tests, two types of configuration of supplementary reinforcement were used, namely two stirrups for the anchorage (marked in red in Figure 2a) and four stirrups for the anchorage (marked in red in Figure 2b). The supplementary reinforcement consisted of 12mm reinforcement and 16mm reinforcement to target reinforcement yielding and concrete dominated failure respectively. The hanger reinforcement (marked in blue in Figure 2a and b) was also provided using the same diameter reinforcing bars as the supplementary reinforcement.

For shear tests, the supplementary reinforcement was provided using 12mm and 20mm diameter rebar as edge reinforcement and stirrups that were placed at a distance of 200 mm from each other (Refer Figure 2c).

![Figure 2: Arrangement of supplementary reinforcement used for tension and shear tests](image-url)

2.3 Test specimen

Normal strength concrete of C20/25 grade was used in the tests. The supplementary reinforcement was provided using deformed steel bars with characteristic yield strength of 500 MPa. The average cubic concrete strength was measured as 33.5 MPa for the tension tests and 31.0 MPa for the shear tests. The mean yield strength for the reinforcing bars was measured as 540 MPa.

2.4 Test Setup

The test setup used for the tension and shear tests are shown in Figure 3. The tests were performed with monotonically increasing force at quasi-static loading rate. The load, displacement and the strains in the reinforcing bars were recorded at a frequency of 5 Hz. In shear tests, no restraint was used to prevent uplift of the anchorage.
3 Test results

3.1 Tests on anchorages under tension forces

3.1.1 Anchorages without supplementary reinforcement

The load-displacement curves and the typical failure mode obtained from the tension tests on anchor groups without supplementary reinforcement are given in Figure 4. The peak load is reached at a displacement of approx. 0.5 mm beyond which a steep descending branch of the load-displacement response can be observed in all the three tests, typical for concrete breakout failure in unreinforced concrete, which was also the observed failure mode in the tests. The mean value of the failure load was recorded as 256.9 kN.

Figure 4: Load-displacement curves and typical failure mode obtained from the tension tests on anchor groups without supplementary reinforcement
3.1.2 Anchorage with supplementary reinforcement, $d_s = 10\text{mm}$

Figure 5 presents the load-displacement curves for the anchorages with $d_s=10\text{mm}$ supplementary reinforcement of type 1 and type 2 (refer to Figure 2) tested under tension loads. Although three tests were performed for each case, due to the problems in the test setup, only one test for supplementary reinforcement Type 1 and two tests for supplementary reinforcement Type 2 could be successfully performed. For comparison, the load-displacement curves from the tests on anchorages without supplementary reinforcement are also plotted in the same graph. In one test on anchorage with Type 2 reinforcement, the displacement measurement was not accurate. For other tests, it can be noticed that the initial stiffness for the anchorages without and with supplementary reinforcement are practically equal. The peak failure load in case of the test with Type 1 reinforcement was obtained as 361.2 kN and the mean failure load for the tests performed on anchorages with Type 2 reinforcement was 460.0 kN. The peak failure loads in case of tests with supplementary reinforcement are reached at much higher displacements compared to the tests without supplementary reinforcement and a ductile load-displacement behavior was observed.

![Load-displacement curves and typical failure mode obtained from the tension tests on anchor groups with supplementary reinforcement, $d_s=10\text{mm}$](image)

In case of the test on anchorage with supplementary reinforcement of Type 1 as well as Type 2, the supplementary reinforcement ruptured due to excessive elongations signifying full mobilization of the stirrups after cracking of concrete.

3.1.3 Anchorage with supplementary reinforcement, $d_s = 16\text{mm}$

The load-displacement curves obtained from the tension tests performed on the anchorages with $d_s=16\text{mm}$ supplementary reinforcement of type 1 and type 2 (refer to Figure 2) tested under tension loads are shown in Figure 6. Three tests were successfully performed for each case. For comparison, the load-displacement curves from the tests on anchorages without supplementary reinforcement are also plotted in the same graph. It can be seen that the initial stiffness of the anchorages without and with supplementary reinforcement are practically equal. The mean peak failure load in case of the tests with Type 1 reinforcement was obtained as 447.6 kN and the mean failure load for the tests performed on anchorages with Type 2 reinforcement was 589.6 kN.
The reinforcement rupture did not occur in any case. In the case of Type 1 reinforcement, concrete breakout was not followed by rebar yielding but bond failure of stirrups occurred. In case of Type 2 reinforcement, the failure mode is classified as concrete strut failure.

3.2 Tests on anchorages under shear forces

3.2.1 Anchorages without supplementary reinforcement

Figure 7 presents the load-displacement curves for the anchorages without supplementary reinforcement loaded in shear towards and perpendicular to the edge. The peak load is reached at a displacement of approx. 1.5mm followed by a relatively steep descending branch typical for concrete edge failure. The mean value of the failure load was recorded as 161.7 kN.
3.2.2 Anchorage with supplementary reinforcement, $d_s = 12\text{mm}$

The load-displacement curves and the typical failure mode obtained from the shear tests performed on the anchorages with supplementary reinforcement of $d_s=12\text{mm}$ are plotted in Figure 8. For comparison, the load-displacement curves from the tests on anchorages without supplementary reinforcement are also plotted in the same graph. It can be again observed that the initial stiffness of the load-displacement curves for groups with supplementary reinforcement and the groups without supplementary reinforcement is similar. For tests on anchorages with supplementary reinforcement, the mean failure load was recorded as 263.4 kN. In the case of anchorages with supplementary reinforcement, after reaching the peak load, the load-displacement curves display a comparatively ductile behaviour. Thus, the presence of even a small amount of the supplementary reinforcement increased not only the load-carrying capacity but also the deformation capacity of the anchor groups. The failure was attributed to reinforcement yielding.

![Figure 8: Load-displacement curves and typical failure mode obtained from the shear tests on anchor groups with supplementary reinforcement, $d_s=12\text{mm}$](image)

3.2.3 Anchorage with supplementary reinforcement, $d_s = 20\text{mm}$

The load-displacement response and the typical failure mode for the anchorages with supplementary reinforcement of $d_s=20\text{mm}$ is plotted in Figure 9 along with the results of the tests without supplementary reinforcement and with supplementary reinforcement of $d_s=12\text{mm}$. A relatively sharp degradation of the response in the post-peak range is observed, which is attributed to failure of the concrete in pryout as shown in the typical failure mode. The mean peak failure load for the tests on anchorages with supplementary reinforcement of diameter $d_s=20\text{mm}$ is obtained as 328.8 kN.

It can be observed that when a small amount of supplementary reinforcement is added, which results in reinforcement failure, a significant increase in the failure load as well as a ductile load-displacement response can be obtained. However, on increasing the amount of supplementary reinforcement, beyond a certain point, the concrete pryout failure started to govern the failure of the anchorage. Consequently, the increase in the failure loads is rather limited and the load-displacement response is also not ductile.
Evaluation of test results

In tests on anchorages with supplementary reinforcement, strain gauges were provided at the locations where the theoretically anticipated failure crack would intercept the supplementary reinforcement. On each location, two strain gauges were installed on the reinforcing bars. The strains recorded by the strain gauges were averaged and converted into stress in the reinforcing bar assuming an elastic-perfectly plastic stress-strain curve with a yield stress of 540 MPa. The stress in the reinforcing bar was multiplied by the cross-sectional area to obtain the tensile force carried by the rebar. The tensile forces carried by all the reinforcing bars intercepted by the crack were added up to obtain the total tension force carried by the activated stirrups. In case of tension tests, the total tension force carried by the stirrups was deducted from the total applied force to calculate the contribution of concrete. In case of shear tests, the total tension force carried by the stirrups was converted into the contribution of the stirrups to carry applied shear force considering the lever arm between the stirrups and the applied force. This shear force was then deducted from the applied shear force to obtain the contribution of concrete to carry shear loads.

4.1 Tests under tension loads

Figure 10 displays the segregated tension force carried by stirrups and concrete as a function of the displacement for anchorages tested under tension loads with Type 1 supplementary reinforcement of (a) $d_s=10\text{mm}$ and (b) $d_s=16\text{mm}$. In both the cases, initially the plot of total force is identical with the plot of the force carried by concrete. Until this point the reinforcement carries a negligible force. However, once the contribution of concrete reaches a value close to the failure load in unreinforced concrete, the reinforcement contribution starts increasing significantly while the concrete contribution drops down. In the case of anchorages with low amount of supplementary reinforcement, at peak load, the stirrups have yielded and the concrete contribution is less than the failure load of the anchorage measured in unreinforced concrete. For anchorages with high amount of supplementary reinforcement, once the contribution of concrete reaches a value close to the capacity of the group in unreinforced concrete, the reinforcement contribution increases significantly.
while the concrete contribution drops down. In this case, the reinforcement yielding was not observed from the strain gauge data. Nevertheless, in every test at peak load, the reinforcement reaches its maximum contribution and the concrete contribution is less than the peak failure load for the anchorage in unreinforced concrete but is not zero (as considered in the EN1992-4 model).

Figure 10: Segregated contributions of concrete and stirrups in carrying the total tension force in case of tests performed on anchorages under tension loads with reinforcement Type 1, for the cases of (a) \(d_s=10\text{mm}\) and (b) \(d_s=16\text{mm}\)

### 4.2 Tests under shear loads

Figure 11 displays the segregated shear force carried by stirrups and concrete as a function of the displacement for anchorages with supplementary reinforcement of \(d_s=12\text{mm}\) tested under shear loads. Again, initially the concrete carries all the applied force and the reinforcement contribution is negligible. Once the contribution of concrete reaches a value close to the concrete edge failure load in unreinforced concrete, the reinforcement contribution starts increasing significantly while the concrete contribution drops down.

Figure 11: Contributions of concrete and stirrups in carrying the total shear force and the development of tension forces carried by the stirrups in case of tests performed on anchorages under shear loads with reinforcement \(d_s=12\text{mm}\)
The peak load is reached when the stirrups have yielded and the concrete contribution of the order of 50% of the failure load of the anchorage measured in unreinforced concrete. The results of tension forces carried by individual stirrups show that at first the two stirrups closest to the anchorage take up tension forces (see plots of F2 and F3), while the effectiveness of the other two stirrups (F1 and F4) is rather low. However, once the stirrups close to the anchors reach almost yielding, the next two stirrups start carrying more forces (see plots of F1 and F4). This shows that the stirrups first intercepted by the crack are the most effective, while the effectiveness of the stirrups next intercepted by the crack depends on the yielding or non-yielding of the stirrups before.

Figure 12 displays the segregated shear force carried by stirrups and concrete as a function of the displacement for anchorages with supplementary reinforcement of $d_s=20\text{mm}$ tested under shear loads. The behavior is rather similar to the case of anchorage with low amount of supplementary reinforcement. Initially, the entire shear force is carried by concrete only. Once the contribution of concrete reaches a value close to the capacity of the group in unreinforced concrete, the reinforcement contribution starts increasing significantly while the concrete contribution drops down. Again, it can be seen that initially the stirrups closest to the anchorage that are first intercepted by the crack start to take up the shear forces (F2 and F3). The other stirrups (F1 and F4) get activated when the crack reaches them. However, in this case, none of the stirrups yielded (yield capacity of one stirrup = 170 kN). This might be attributed to the initiation of a pryout failure mode as observed in the tests which limited the capacity of the anchorage.

Figure 12: Contributions of concrete and stirrups in carrying the total shear force and the development of tension forces carried by the stirrups in case of tests performed on anchorages under shear loads with reinforcement $d_s=20\text{mm}$

In the existing models of EN1992-4\textsuperscript{1}, ACI318\textsuperscript{2} as well as fib Bulletin 58\textsuperscript{3}, the failure load is considered as the greater of either the failure load in unreinforced concrete or the force carried by reinforcement only. The evaluation of the test results show that at peak load, the contribution of concrete is not zero but is significant. Ignoring the contribution of concrete makes the current models quite conservative.
5 Conclusions

The test results presented in this work clearly show that supplementary reinforcement in the form of surface (or edge) reinforcement and stirrups increases significantly the capacity of anchorages loaded in tension (or shear close and towards the edge). Even a relatively small amount of supplementary reinforcement leads to a significant increase in the failure loads of the anchorage. This is in contrast with the assumptions currently followed in the standards which consider that until the reinforcement contribution alone exceeds the anchorage capacity in unreinforced concrete, the failure load of the anchorage remains equal to its capacity in unreinforced concrete. This assumption definitely makes the models given in current standards relatively conservative for both tension and shear.

Under shear loads, another aspect which was highlighted clearly in this work was the development of failure crack. In all the tests on anchorages in unreinforced concrete as well as with supplementary reinforcement, the failure crack appeared from the back anchor row. The model in EN1992-4\(^1\) considers failure crack originating from the front anchor row. This results in smaller concrete capacity as well as small anchorage lengths of the stirrups leading to a smaller stirrup contribution. Both these aspects make the model given in EN1992-4 very conservative for anchorages with more than one anchor row.

However, the capacity of an anchorage under tension or shear loads cannot be indefinitely increased by addition of supplementary reinforcement since in case of high amounts of supplementary reinforcement, the failure is limited by the concrete failure modes such as strut failure or pryout failure.

The comparison of the test results with existing models and a new model for anchorages with supplementary reinforcement is presented in accompanying paper.

6 Acknowledgements

The tests reported in this work were funded by Peikko Group Corporation. The support and efforts of Mr. Jan Bujnak, Peikko is greatly appreciated.

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COMPREHENSIVE ANALYTICAL MODEL FOR ANCHORAGES WITH SUPPLEMENTARY REINFORCEMENT

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ABSTRACT

The models included in the current standards and guidelines1,2,3 to evaluate the failure loads for anchorages with supplementary reinforcement subjected to either tension or shear forces are very conservative. The existing models do not explicitly consider all three major components that provide resistance to applied forces for anchorages with supplementary reinforcement namely the concrete struts, the tension ties and the nodes. The models in EN1992-41 and fib Bulletin 583 consider the failure of tension tie (stirrups) due to bond or yielding while giving an indirect and very conservative consideration to the node (hooks) and ignore the possibility of strut failure. ACI 3182 considers only yielding of the stirrups provided sufficient anchorage length is available within and outside the breakout body, while not considering strut failure. For anchorages under shear forces, the model in EN1992-4 considers the failure crack to originate from the front row of the anchors, which leads to a small reinforcement resistance making the model very conservative. The model in ACI and fib consider the failure crack to originate from the back anchor row however in that case the anchor steel failure (as well as the pryout failure) is calculated considering that the shear load is taken up by only the anchors in the back anchor row as well. Nevertheless, in all the models, the resistance for the concrete breakout and the reinforcement are not combined but only the higher of the two resistances is considered as the failure load of the anchorage. These assumptions make the existing models quite conservative for many cases. However, on the other hand, due to the fact that the possibility of the strut failure is not explicitly considered, for the cases of anchorages with high amounts of supplementary reinforcement, the existing models tend to get unconservative.

In this work, a new analytical model is developed based on detailed experimental investigations performed on anchorages with supplementary reinforcement to take up tension forces subjected to tension loads and anchorages with supplementary reinforcement to take up shear forces close to an edge loaded in shear perpendicular and towards the edge. The model gives due consideration to and explicitly considers all three components of resistance provided by the anchorage with supplementary reinforcement. The model combines and synergizes the strength of the models proposed based on research performed at the University of Stuttgart4,5. The mean failure loads calculated by the new model are shown to be in very good agreement with the experimentally obtained mean failure loads. The model is objective and is applicable equally well for anchorages with supplementary reinforcement under either tension or shear forces.
1 Introduction

In an accompanying paper, systematic investigations made on quadruple anchorages (2x2 anchor groups), in the form of WELDA® anchor plates from Peikko Group Corporation, without and with supplementary reinforcement under tension loads and under shear loads close and towards the edge are presented. Different amounts of supplementary reinforcement were used in the tests targeting the reinforcement and concrete failure modes. It was shown that the failure load for anchorages under tension or shear close and towards the edge can be significantly increased by using supplementary reinforcement. In case of anchor groups with supplementary reinforcement, once the concrete cracks, the stirrups get activated and provide resistance to the applied tension (or shear) loads until reinforcement yielding or bond failure occurs. Thus, the tension or shear strength of the anchorage can be increased by increasing the amount of supplementary reinforcement. However, this increase is limited by concrete based failure modes such as strut failure or pryout failure in case of anchorages provided with high amount of supplementary reinforcement.

In the current standards, such as EN1992-4, ACI 318 and fib bulletin 58, a very conservative approach is given to consider the influence of the supplementary reinforcement on failure loads of the anchorages. In this paper, the test results are compared with the existing models as given in EN1992-4 for anchorages with supplementary reinforcement loaded in tension or shear towards the edge. Further, a new model is proposed for predicting the failure loads for anchorages with supplementary reinforcement undergoing reinforcement failure by modifying the model proposed by Schmid for anchorages with supplementary reinforcement loaded in shear towards the edge and the model proposed by Berger to consider the possible strut failure in case of anchorages with high amount of supplementary reinforcement. Additionally, a new approach is presented to calculate pryout failure loads for anchor groups with more than one anchor row when the failure crack for concrete edge failure is assumed from the back anchor row. It is shown that with the proposed model, the failure loads for the low to high amount of reinforcement (where reinforcement failure dominates) can be predicted very well.

2 Model given in EN1992-4

According to EN1992-4, for anchorages with supplementary reinforcement, the load corresponding to failure of reinforcement in the concrete breakout body can be obtained on the basis of the strut-and-tie model (Figure 1). In EN1992-4, only bars with a distance ≤ 0.75 times the embedment depth (for tension loads, see Figure 1a) or the edge distance (for shear loads, see Figure 1b) from the fastener are assumed as effective.
The stirrups are considered effective provided the anchorage length $l_1$ (Figure 1) in the concrete breakout body is at least equal to 10 times the rebar diameter (straight bars) or at least equal to 4 times the rebar diameter (bars with a hook, bend or loop).

As per EN1992-4 (2015), the characteristic resistance, $N_{Rk, re}$ of the supplementary reinforcement provided for one fastener associated with anchorage failure in the concrete breakout body is given by:

$$N_{Rk, re} = \sum_{n} N_{n, re}^0$$  \hspace{1cm} (1)

With, $n =$ number of legs of the anchor reinforcement effective for one fastener

$$N_{n, re}^0 = \frac{l_1 \cdot \pi \cdot d_{s, re} \cdot f_{bk}}{\alpha_1 \cdot \alpha_z} \leq f_{yk} \cdot A_{s, re}$$  \hspace{1cm} (2)

Where,

$l_1 =$ anchorage length = distance from the intersection of theoretical crack and the rebar to the stirrup end (Figure 1)

$d_{s, re} =$ diameter of the stirrup

$f_{bk} =$ characteristic bond strength = $1.5 f_{bd}$

$f_{bd} =$ design bond strength according to EN1992-1-1

$f_{yk} =$ characteristic yield strength of reinforcing bars

$A_{s, re} =$ Area of reinforcing bar used as stirrup

$\alpha_1 =$ influencing factor that assumes a value of 0.7 for hooked rebar if $c_d < 3d_{sre}$ or 1.0 if $c_d \geq 3d_{sre}$ and 1.0 for straight rebar

$\alpha_2 =$ factor to consider the influence of cover on the bond strength defined for hooked bars as

$$\alpha_2 = 1 - 0.15 \left( c_d - 3d_{sre} \right) d_{s, re} ; \text{ with } 0.7 \leq \alpha_2 \leq 1.0$$

$c_d =$ clear cover to the reinforcing bar in any direction or half the clear distance to the adjacent rebar, whichever is smaller

For tension loads, Eq (1) gives the load corresponding to supplementary reinforcement failure. For shear loads, this load is converted to shear loads considering the eccentricity between the applied shear force and stirrups as

$$V_{Rk, re} = \frac{N_{Rk, re}}{x}$$  \hspace{1cm} (3)

Where, $x$ is the factor to consider for the eccentricity between the reinforcement and the applied shear load (compare Figure 1b)

$$x = \left( 1 + \frac{e_s}{z} \right)$$

$e_s =$ distance between reinforcement and shear force acting on a fixture

$z =$ internal lever arm of the concrete member that is approx. equal to $0.85d$

$d =$ min(depth of concrete member, $2h_{ef}, 2c_1$)
The characteristic failure load for the anchorage with supplementary reinforcement under tension (or shear) loads, is given as

\[ N_{Rk} = \max \left( N_{Rk,c} ; N_{Rk,se} \right) ; \quad V_{Rk} = \max \left( V_{Rk,c} ; V_{Rk,se} \right) \]

Where,

\[ N_{Rk,c} = \text{Characteristic resistance corresponding to concrete cone failure (under tension) of an anchorage without supplementary reinforcement} \]
\[ V_{Rk,c} = \text{Characteristic resistance corresponding to concrete edge failure (under shear) of an anchorage without supplementary reinforcement} \]

The mean resistances can be obtained by multiplying Eq. (2) by 1.33\(^8\).

From Eq. (2), it implies that if sufficient cover is available, the hook resistance is considered as around 40% of the bond resistance of the stirrups. Thus, longer the bond length, higher is the hook resistance as well, which does not seem logical intuitively. This aspect was highlighted and targeted in the model by Schmid\(^4\) as will be discussed later.

3 Comparison of test results with the model of EN1992-4

3.1 Tension tests

The comparison of the mean failure loads obtained from the experiments with the mean failure loads calculated as per the model of EN1992-4\(^1\) for the tension tests are given in Table 1. As described in the accompanying paper\(^6\), two different configurations of supplementary reinforcement against tension forces were tested, with 2 stirrups only outside the anchorage (4 stirrup legs in total) and with 2 stirrups outside and two within the anchorage (8 stirrup legs in total). These configurations were named as Type 1 and Type 2 reinforcement configuration respectively.

<table>
<thead>
<tr>
<th>Diameter of supplementary reinforcement</th>
<th>Type of reinforcement</th>
<th>Total c.s. area of stirrup legs [mm(^2)]</th>
<th>Mean failure loads [kN]</th>
<th>Experiment</th>
<th>EN1992-4</th>
<th>Ratio N(<em>{u,exp}/N(</em>{u,calc})</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (Unreinforced)</td>
<td>NA</td>
<td>0</td>
<td></td>
<td>256.9</td>
<td>295.2</td>
<td>0.87</td>
</tr>
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<td>10 mm</td>
<td>Type 1</td>
<td>314</td>
<td>361.2</td>
<td>295.2</td>
<td>1.22</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type 2</td>
<td>628</td>
<td>460.0</td>
<td>295.2</td>
<td>1.56</td>
<td></td>
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<td>Type 1</td>
<td>804</td>
<td>447.6</td>
<td>295.2</td>
<td>1.52</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type 2</td>
<td>1608</td>
<td>589.6</td>
<td>295.2</td>
<td>2.00</td>
<td></td>
</tr>
</tbody>
</table>

The mean tension failure predicted by the model of EN1992-4\(^1\) for the anchorage in unreinforced concrete is slightly on the unconservative side, which can be attributed to the scatter of the failure loads. However, for the cases with supplementary reinforcement, while the tests show a clear increase in the failure loads but the model in EN1992-4\(^1\) predicts no increase in the failure loads. This is due to the fact that the reinforcement resistance never exceeds the concrete cone resistance in unreinforced concrete for the tested cases. The results clearly bring out the conservatism in the existing model of EN1992-4\(^1\).
3.2 Shear tests
Table 2 presents the comparison of the mean failure loads obtained from the experiments with the mean failure loads calculated as per the model of EN1992-4\(^1\) for the shear tests. It can be seen that the model for concrete edge failure under shear loads is even more conservative than the model for concrete cone failure under tension loads. This is due to the fact that EN1992-4 considers the failure load to be evaluating considering the failure crack from front anchors. Even for the case of anchorage without supplementary reinforcement, the model given in EN1992-4 is quite conservative. Again, as per the model the reinforcement resistance does not exceed the concrete edge resistance in unreinforced concrete for the tested cases.

Table 2: Comparison of mean failure loads for shear tests

| Diameter of supplementary reinforcement | C.S. area of one stirrup leg [mm\(^2\)] | Mean shear failure loads [kN] | | |
|----------------------------------------|---------------------------------------|-------------------------------|---|
| 0 (Unreinforced)                       | 0                                     | 161.7                         | 60.2 | 2.69 |
| 12 mm                                  | 113                                   | 263.4                         | 60.2 | 4.38 |
| 20 mm                                  | 314                                   | 328.8                         | 60.2 | 5.46 |

4 New model

4.1 Reinforcement failure
The model for reinforcement failure proposed here is a modified version on the model developed by Schmid\(^4\) for anchorages with supplementary reinforcement loaded under shear loads. Schmid\(^4\) proposed that the anchorage capacity of the stirrup leg is given by the sum of the hook capacity and the bond capacity. As per Schmid\(^4\), the hook capacity is relatively independent of the bond length of the stirrup in the breakout body. The original model by Schmid\(^4\) has been modified considering the observations made in the tests as well as to make the model suitable for tension loads as well.

For anchorages in unreinforced concrete, in general, the new model uses the same formulations as given in EN1992-4 to calculate the failure loads. However, under shear loads close and towards the edge, it is assumed in the proposed model that the failure loads are always calculated from the back anchor row both in case of anchorage without and with supplementary reinforcement. Further, the load carrying capacity of the anchor reinforcement consists of two parts: the contribution of hook and the contribution of bond.

The mean anchorage capacity of one stirrup leg, \(N^{0}_{Rm,re}\) is given as

\[
N^{0}_{Rm,re} = N^{0}_{Rm,hook} + N^{0}_{Rm,bond} = A_{r, re} f_{ym} \tag{4}
\]

Where, \(A_{s, re}\) = area of one stirrup leg and \(f_{ym}\) = mean yield strength of stirrup

The stirrups that enclose the surface reinforcement (for tension loads) or the edge reinforcement (for shear loads) are considered as effective provided they have a bond length of at least \(4d_s\) within the breakout body. The mean value of hook contribution for a particular stirrup leg, \(N^{0}_{Rm,hook}\) is given as
\[ N_{um, hook}^0 = \psi_1 \cdot \psi_2 \cdot \psi_3 \cdot A_y \cdot f_{ym} \cdot \left( \frac{f_{cm, cube}}{30} \right)^{0.1} \]  

(5)

Where, the factor \( \psi_1 \) considers the influence of the position of the stirrup. A value of \( \psi_1 = 0.95 \) is assumed for stirrups that either lie between the outermost anchors or, if they lie outside the anchorage, are first intercepted by the crack. If yielding of the stirrups first intercepted by the crack takes place, the next stirrup is assigned a value of effectiveness factor, \( \psi_{1,2} = 0.95 \) else, \( \psi_{1,2} = 0.16 \).

The factor \( \psi_2 \) considers the influence of the diameter of the surface reinforcement (for tension) or edge reinforcement (for shear), \( d_{s,L} \) with respect to the diameter of the stirrup, \( d_{s, re} \):

\[ \psi_2 = \left( \frac{d_{s,L}}{d_{s, re}} \right)^{0.25} \leq 1.2 \]  

(6)

The factor \( \psi_3 \) considers the influence of the bond length, \( l_1 \), of the stirrup in the breakout body and its diameter given as:

\[ \psi_3 = \left( \frac{l_1}{h_{ef}} \right)^{0.4} \cdot \left( \frac{10}{d_{s, re}} \right)^{0.25} \quad \text{for tension loads} \]  

(7)

\[ \psi_3 = \left( \frac{l_1}{c_{1,n}} \right)^{0.4} \cdot \left( \frac{10}{d_{s, re}} \right)^{0.25} \quad \text{for shear loads} \]  

(8)

Where, \( h_{ef} \) is the effective embedment depth of the anchors (in case of tension loads) and \( c_{1,n} \) is the edge distance of the back anchor row (in case of shear loads).

The contribution of the bond of one stirrup leg is given as:

\[ N_{Rm, bond}^0 = \pi \cdot d_{s, re} \cdot (l_1 - l_{min}) \cdot f_{bm} / \alpha_2 \]  

(9)

Where, \( l_{min} \) is the minimum anchorage length required (=4\( d_{s, re} \)), \( f_{bm} \) is the mean bond strength (=1.33\( f_{bck} \))

\( \alpha_2 = \text{factor to consider the influence of cover on bond strength defined as} \quad \alpha_2 = 1 - 0.15(c_d - d_{s, re}) / d_{s, re} \)

\( c_d = \text{clear cover to the stirrup leg in any direction or half the clear distance to the adjacent stirrup, whichever is smaller} \)

The total capacity of the anchor reinforcement is calculated by summing up the capacities of all effective stirrup legs

\[ N_{Rm, re} = \sum N_{Rm, re}^0 \]  

(10)
\( n = \text{number of effective stirrup legs of the anchorage. Effective are stirrups with an anchorage length,} \ l_i \geq 4d_{s, re} \text{ in the theoretical breakout body.} \)

Under tension, Eq. (10) gives the contribution of the supplementary reinforcement under tension loads. However, to obtain the contribution of the supplementary reinforcement towards the shear loads, Eq. (3) is used to convert the tension resistance of the reinforcement to the shear resistance.

The evaluation of the test results\(^6\) highlighted that the concrete carries a significant percentage of the failure load corresponding to concrete cone (or edge) failure in unreinforced concrete when the peak load of the anchorage is reached. Based on the evaluation of the test results and to keep the model simple, it seems reasonable to assume that at peak load, the load taken up by the concrete is approx. 50\% of the concrete cone (or edge) failure load of the anchorage without supplementary reinforcement. Therefore, it is proposed that the peak failure load of an anchorage with supplemental reinforcement is given as the failure load corresponding to 50\% of the concrete cone (or edge) failure load in unreinforced concrete plus the load corresponding to reinforcement failure calculated in accordance with the new model. Thus the mean tension resistance of an anchorage with supplementary reinforcement is given by

\[
N_{\text{Rm}} = 0.5 \cdot N_{\text{Rm,c}} + N_{\text{Rm, re}} \geq N_{\text{Rm,c}}
\]

Similarly, the mean shear resistance of an anchorage loaded towards the edge with supplementary reinforcement is given by

\[
V_{\text{Rm}} = 0.5 \cdot V_{\text{Rm,c}} + V_{\text{Rm, re}} \geq V_{\text{Rm,c}}
\]

\subsection{4.2 Strut failure}

An upper limit to the tension failure load of the anchorage with supplementary reinforcement applies due to a possible strut failure. This failure mode was investigated by Berger\(^5\) by performing equivalent tests on anchorages in unreinforced concrete with varying the position of the supports. According to the model by Berger\(^5\), the maximum possible failure load of an anchorage with anchor reinforcement compared to the same anchorage in unreinforced concrete is given by Eq. (13).

\[
N_{\text{um, max}} = \psi_{\text{strut}} \cdot N_{\text{Rm,c}}
\]

where, \( \psi_{\text{strut}} \) is the strut factor which depends on the anchorage and stirrup configuration.

The basic strut factor is defined as

\[
\psi_{\text{strut}}^0 = 2.75 - 1.17 \cdot \frac{x}{h_{ef}} \geq 1.0
\]

\( x = \text{distance between the secondary failure cone on the concrete surface and the anchor axis (see Figure 2). Eq. (14) is valid for stirrups that enclose the surface reinforcement.} \)
For single anchor with one stirrup each at both sides of the anchor, the strut factor, $\psi_{\text{strut}}$ is equal to the basic strut factor, $\psi_{\text{strut}}^0$. For the case of anchor groups with supplementary reinforcement, Berger proposes that the factor for strut failure should be calculated considering the contributory area of the anchorages.

For anchorages with stirrup legs arranged symmetrically outside the anchorage (Type 1), the maximum failure load considering strut failure is given as

$$
\psi_{\text{strut}} = \psi_{\text{strut}}^0 \times \left( \frac{A_{c,N,1}}{A_{c,N}} + \frac{A_{c,N,2}}{A_{c,N}} + \psi_{\text{strut}}^0 \times \frac{A_{c,N,3}}{A_{c,N}} \right)
$$

For anchorages with two stirrup legs arranged symmetrically outside the anchorage (Type 2), the maximum failure load considering strut failure is given as

$$
\psi_{\text{strut}} = \psi_{\text{strut}}^0 \times \left( \frac{A_{c,N,1}}{A_{c,N}} + \frac{A_{c,N,2}}{A_{c,N}} + \frac{A_{c,N,3}}{A_{c,N}} \right)
$$

Figure 2: Strut-and-tie Model according to Berger for anchorage with supplementary reinforcement

Figure 3: Consideration of strut formation in case of anchor groups with supplementary reinforcement under tension forces
For the case of anchorages with supplementary reinforcement also lying within the outermost anchor rows and arranged symmetrically to the anchors (Type 2), the factor $\psi_{\text{strut}}$ is given as:

$$
\psi_{\text{strut}} = \psi_{\text{strut}}^0(x) \cdot \left( \frac{A_{e,N,1} + A_{e,k}}{A_{e,N}} \right) + \psi_{\text{strut}}^0(x) \cdot \left( \frac{A_{e,N,2} + A_{e,N,3}}{A_{e,N}} \right)
$$

(16)

A model analogous to the model shown here is also applicable for anchorages with supplementary reinforcement under shear loads. However, due to the space limitations and the fact that in the tests under shear loads, strut failure did not occur due to the precedence by pryout failure, it is not discussed here.

### 4.3 Pryout failure

As described in the accompanying paper\(^6\), in case of tests with high amount of supplementary reinforcement under shear loads, the reinforcement failure was preceded by the pryout failure. Please note that this failure could be avoided by using a restraint against uplift of the base plate as was used in another similar test program by the authors\(^9\). In such cases the concrete contribution is not 50% of its capacity in unreinforced concrete as used in Eq. (12) but 100%, even at the point of reinforcement yielding. Nevertheless, it is impractical or unreliable to have such a constraint in reality, therefore in this test program, the uplift restraint was not used.

The current approach to evaluate the failure load of an anchorage corresponding to pryout failure, $V_{Rm,cp}$ involves multiplying the tension failure load of the anchorage by a factor, $k_3$. This model considers the resistance associated with concrete breakout as well as the ratio between the tension force in the anchor and the applied shear force\(^8\).

$$
V_{Rm,cp} = k_3 \cdot N_{Rm,c}
$$

(17)

Where,

- $V_{Rm,cp}$ is the mean pryout failure load for the anchorage
- $N_{Rm,c}$ is the mean tension failure load for the anchorage
- $k_3$ is the factor that assumes the value
  - $k_3 = 1.0$ for $h_{ef} \leq 60\text{mm}$
  - $k_3 = 2.0$ for $h_{ef} > 60\text{mm}$

As per the current standards\(^1,3\), for the anchorages close to an edge and loaded in shear towards the edge, if the concrete edge failure load is evaluated assuming the failure crack from the front anchor row, all anchors are considered to evaluate the pryout failure load (approach given in EN1992-4\(^1\)). However, if the load corresponding to concrete edge failure is evaluated assuming the failure crack from back anchor row then only the back anchors are considered to evaluate the pryout failure load (approach given in fib\(^3\)).
In the new model, the concrete edge failure loads are calculated assuming the failure crack from back anchor row. However, if only the back anchor row is considered to calculate pryout failure load, the failure loads of the anchorages with two or four anchor rows would be essentially equal. This concept is explained in Figure 4a. However, the results of a few tests additionally performed on anchorages with four anchor rows (not presented here) clearly show that the failure loads for anchorages with four anchor rows could reach much higher values than the loads which induced pryout failure in anchorages with two anchor rows with supplementary reinforcement. Therefore, the approach of considering only the anchors in the back row to calculate pryout failure loads is unobjective.

The test results displayed that in the cases where pryout failure occurred, a major crack appeared from the front anchor row. Therefore, this anchor could not take up major shear load. Therefore, it is proposed that the pryout failure load for an anchorage with supplementary reinforcement is evaluated considering the anchor group formed by all anchor rows except the front anchor row and considering a free edge at the line of the front anchor row. This method is explained in Figure 4b.
5 Comparison of test results with the new model

Using the formulations given above, the mean failure loads for the anchorages as obtained following the new model were compared with the experimentally obtained mean failure loads for the anchorages tested.

5.1 Tension loads

The comparison of the mean failure loads obtained from the experiments with the mean failure loads calculated as per the new model for the tension tests are given in Table 3. For comparison, the failure loads calculated as per the model of EN1992-4\(^1\) are also included in Table 3.

<table>
<thead>
<tr>
<th>Diameter of supplementary reinforcement</th>
<th>Type of reinforcement</th>
<th>Total c.s. area of stirrup legs [mm(^2)]</th>
<th>Mean tension failure loads [kN]</th>
<th>Experiment</th>
<th>EN1992-4</th>
<th>New model</th>
<th>(\frac{N_{u,exp}}{N_{u,calc,EN1992-4}})</th>
<th>(\frac{N_{u,exp}}{N_{u,calc,NewModel}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (Unreinforced)</td>
<td>NA</td>
<td>0</td>
<td>256.9</td>
<td>295.2</td>
<td>295.2</td>
<td>0.87</td>
<td>0.87</td>
<td></td>
</tr>
<tr>
<td>10 mm</td>
<td>Type 1</td>
<td>314</td>
<td>361.2</td>
<td>295.2</td>
<td>317.2</td>
<td>1.22</td>
<td>1.14</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type 2</td>
<td>628</td>
<td>460.0</td>
<td>295.2</td>
<td>486.9</td>
<td>1.56</td>
<td>0.94</td>
<td></td>
</tr>
<tr>
<td>16 mm</td>
<td>Type 1</td>
<td>804</td>
<td>447.6</td>
<td>295.2</td>
<td>500.4</td>
<td>1.52</td>
<td>0.89</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Type 2</td>
<td>1608</td>
<td>589.6</td>
<td>295.2</td>
<td>566.6</td>
<td>2.00</td>
<td>1.04</td>
<td></td>
</tr>
</tbody>
</table>

*The failure load is controlled by reinforcement failure.  
# The failure load is controlled by strut failure

It can be seen that for the cases with supplementary reinforcement, the new model gives much better predictions compared to the model in EN1992-4\(^1\). The new model predicts an increase in the failure loads even with relatively low amounts of supplementary reinforcement, same as in the experiments. Furthermore, due to the consideration of strut failure, the failure loads calculated by the new model are in close agreement with the experiments also for high amounts of supplementary reinforcement. The failure modes predicted by the new model are the same as observed in the experiments\(^5\).

5.2 Shear loads

Table 4 presents the comparison of the mean failure loads obtained from the experiments with the mean failure loads calculated as per the new model for the shear tests.

<table>
<thead>
<tr>
<th>Diameter of supplementary reinforcement</th>
<th>C.S. area of one stirrup leg [mm(^2)]</th>
<th>Mean shear failure loads [kN]</th>
<th>Experiment</th>
<th>EN1992-4</th>
<th>New Model</th>
<th>(\frac{V_{u,exp}}{V_{u,calc,EN1992-4}})</th>
<th>(\frac{V_{u,exp}}{V_{u,calc,NewModel}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (Unreinforced)</td>
<td>0</td>
<td>161.7</td>
<td>60.2</td>
<td>142.5</td>
<td>2.69</td>
<td>1.13</td>
<td></td>
</tr>
<tr>
<td>12 mm</td>
<td>113</td>
<td>263.4</td>
<td>60.2</td>
<td>240.0</td>
<td>4.38</td>
<td>1.10</td>
<td></td>
</tr>
<tr>
<td>20 mm</td>
<td>314</td>
<td>328.8</td>
<td>60.2</td>
<td>315.6</td>
<td>5.46</td>
<td>1.04</td>
<td></td>
</tr>
</tbody>
</table>

*The failure load is controlled by reinforcement failure.  
# The failure load is controlled by pryout failure

The failure loads predicted by the new model for concrete edge failure under shear loads in unreinforced concrete are much more close to the experimentally obtained mean failure loads due to the consideration of failure crack from back anchors. This approach is also recommended by fib Bulletin 58\(^3\) and ACI 318\(^2\) but not in EN1992-4\(^4\).
For the anchorages with supplementary reinforcement, the predicted failure loads are in very good agreement with the experimentally obtained mean failure loads. The new model predicts an increase in the failure load even with relatively small amount of supplementary reinforcement, the same as observed in the experiments. For relatively high amounts of supplementary reinforcement, the failure load is controlled by pryout failure in the new model, which is also the failure mode observed in the tests.

Thus, the new model is able to predict the mean failure loads as well as the failure modes which are in good agreement with the experimentally obtained mean failure loads and the failure modes, both under tension loads as well as shear loads.

6 Conclusions

Based on the evaluation of test results presented in the accompanying paper, a new model is presented in this work to calculate the failure loads for anchorages with supplementary reinforcement. The model considers both the reinforcement and concrete failure modes. The model for the failure of supplementary reinforcement is based on the model proposed by Schmid for anchorages under shear loads and is modified to consider the realistic behavior of the anchorages with supplementary reinforcement as observed from the tests. The model was also modified to make it applicable for anchorages with supplementary reinforcement under tension loads as well. The model for strut failure is based on the model proposed by Berger. A new approach is proposed to calculate the pryout failure loads for the anchorages with more than one anchor row when the failure crack is assumed to appear from the back anchor row. The new model is able to predict the mean failure loads in excellent agreement with the experimentally obtained mean failure loads as well as correctly predict the dominant failure mode, both under tension loads as well as shear loads.

7 Acknowledgements

The tests reported in this work were funded by Peikko Group Corporation. The support and efforts of Mr. Jan Bujnak, Peikko is greatly appreciated.

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2. American Concrete Institute. 2014. ACI 318: Building Code Requirements for Structural Concrete (ACI 318-14).


EXPERIMENTAL ASSESSMENT OF HEADED ANCHORS WITH SUPPLEMENTARY REINFORCEMENT

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ABSTRACT

Headed anchors are an efficient technique to anchor tensile and shear forces to concrete. The load bearing capacity of such anchors is typically limited by the resistance of the surrounding concrete against concrete cone failure. This capacity might be increased by combining the anchors with supplementary reinforcement. A series of tensile tests on headed anchors combined with supplementary reinforcement will be presented in the paper. The tensile tests have been performed on individual headed bars anchored in slender concrete columns. The columns were reinforced by normal and shear reinforcement in order to avoid the global failure of the tested specimen and localize the failure in the anchorage area. The tests demonstrate that under certain conditions, the performance of the anchors is significantly better than predicted by current standards.

1 Introduction

The appropriate design and performance of structural joints are very important preconditions for the proper functioning of precast concrete structures. Currently, several types of anchorage systems are used in precast concrete structures as part of joints. Short headed anchors made of rebar steel represent a relatively popular solution because they provide high resistance while being relatively compact. The technical specification CEN/TS 1992-4-2 defines the design recommendations including methods for the verification of the design value of resistance against concrete cone failure and design recommendations for supplementary steel reinforcement to be provided to prevent such failure. The design approach for concrete cone resistance and design of supplementary reinforcement formulated in CEN/TS 1992-4-2 has been discussed and criticized in several recently published references (Fromknecht 2009, Schmid 2010, Henriques & al. 2013). The results of studies presented in these references indicate that current methods for the assessment of tensile resistances of headed anchors with and without supplementary reinforcement may be extremely conservative for certain specific applications of headed anchors. The purpose of presented research is twofold. Firstly, the experimental investigation demonstrates the resistances and failure modes of short headed anchors with and without supplementary reinforcement. Secondly, the results of experimental test are compared with resistances determined in accordance with the current design methods.
2 Theoretical investigation

The technical specification CEN/TS 1992-4-2\(^1\) defines a set of verifications for different failure modes of headed anchors loaded in tension. The load capacity of an anchor is governed by its geometry, position in concrete member and material properties of concrete and steel. The resistance of an anchorage system is, due to concrete cone failure, obtained from the capacity of one anchor without influence of concrete member edges. In such conditions, the characteristic resistance of this anchor is defined in CEN/TS 1992-4-2\(^1\) as follows:

\[
N_{Rk,c}^0 = k_{cr} \cdot \sqrt{f_{ck,cube} \cdot h_{ef}^{1.5}}
\]  

(2)

According to current experience, the resistance of headed anchor may be increased by using supplementary reinforcement designed in order to prevent the brittle failure by formation of concrete cone. The supplementary steel reinforcement can be used in form of hooks, loops or stirrups. A typical arrangement of a headed anchor with supplementary reinforcement is shown in Figure 1.

According to technical specification CEN/TS 1992-4-2\(^1\), the design value of resistance of an anchor with supplementary reinforcement can be predicted using the following expression:

\[
N_{Rd,a} = \sum_{n} l_{1} \cdot \pi \cdot d_{s} \cdot f_{bd} \cdot \frac{\alpha}{\alpha}
\]  

(2)

where \(l_{1}\) = anchorage length of the supplementary reinforcement in the concrete cone; \(d_{s}\) = diameter of the supplementary reinforcement; \(f_{bd}\) = design bond strength acc. to EN 1992-1-1:2004\(^5\); \(\alpha\) = influence factor (value = 0.7 if supplementary reinforcement is bended in the concrete cone). In formula (2), the resistance of supplementary reinforcement is thus defined by the bond strength of the straight part of the stirrup inside of the concrete cone increased by the effect of the hook end. Such formulation has been demonstrated to be very conservative among other for cases when the stirrup is closed and continuous inside of the concrete cone (see eg. Fromknecht\(^2\)).
3 Experimental investigation

The experimental program was divided into four test series, where the tensile load capacity of headed anchors cast in small concrete members has been tested. Two series of the test program (A,C) are presented in the paper. Single anchors with identical dimensions ($h_{ef} = 140$mm $\phi 25$mm) and material parameters (B500B) have been used in all tested specimens. The test procedure followed the requirements of ETAG 001\(^4\) (2006) for testing of metal anchors for use in concrete. The headed anchors were connected to a hollow hydraulic jack through a steel rod connected to an articulated joint. The load was measured by a force sensor placed on the top of the hydraulic jack, displacements were measured by a pair of displacement sensors (Figure 2).

![Test setup: testing frame, hydraulic jack, force and displacement sensors](image1)

Figure 2: Test setup: testing frame, hydraulic jack, force and displacement sensors

3.1 Test series A

The test series A consisted a total of 5 specimens with different longitudinal and shear reinforcement. The target of this part of the experimental investigation was to demonstrate the influence of stiffness and overall reinforcement arrangement of the specimen. Geometric and material properties of the specimens are presented in Table 1 and Figure 3.

![Test setup: testing frame, hydraulic jack, displacement transducers and force sensor](image2)

Figure 3: Test setup: testing frame, hydraulic jack, displacement transducers and force sensor
Table 1: Geometric and material parameters of specimens – series A

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete column</th>
<th>Column reinforcement</th>
<th>Anchor reinforcement</th>
<th>$N_{\text{test}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$l$</td>
<td>$b$</td>
<td>$h$</td>
<td>$f_{\text{c,cube,m}}$</td>
</tr>
<tr>
<td></td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>MPa</td>
</tr>
<tr>
<td>1A</td>
<td>1500</td>
<td>280</td>
<td>280</td>
<td>41.5</td>
</tr>
<tr>
<td>2A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4A</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>5A</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.2 Test series C

Three specimens with different arrangement of supplementary reinforcement have been tested in this group. The headed anchor in specimen 1C (Fig.4a) was supplemented with reinforcement in form of stirrups and horizontal bars. Anchor in specimen 2C (Fig.4b) was equipped only by stirrups and specimen 3C (Fig.4c) had no anchor reinforcement. The specimens were tested 28 days after casting. Material properties of concrete were determined from cubes 150x150mm cast together with the concrete columns. The dimensions and material properties of test specimens are presented in Table 2.

Figure 4: Specimens 1C, 2C, 3C

Table 2: Geometric and material parameters of specimens – series C

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete column</th>
<th>Column reinforcement</th>
<th>Anchor reinforcement</th>
<th>$N_{\text{test}}$</th>
</tr>
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<td>$l$</td>
<td>$b$</td>
<td>$h$</td>
<td>$f_{\text{c,cube,m}}$</td>
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<td></td>
<td>mm</td>
<td>mm</td>
<td>mm</td>
<td>MPa</td>
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<tr>
<td>1C</td>
<td>1500</td>
<td>280</td>
<td>280</td>
<td>52.5</td>
</tr>
<tr>
<td>2C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3C</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4 Evaluation of the tests

The load displacement behaviour of the tested specimens are presented on Figure 4.

The only specimen in group A where the failure of the anchorage has been observed was the specimen 3A. The failure of the other specimens was associated with the development of diagonal cracks (shear failure of the concrete member), or vertical cracks in the top part of the specimen (bending failure). The reference specimen without supplementary reinforcement 3C failed relatively brittle by the concrete cone failure. The detached concrete cone was bounded by head of anchor in lower part and by longitudinal reinforcement and stirrups in the level of upper reinforcement. The failure of specimen 2C corresponded with cracks and crushing of concrete between the anchor and the supplementary reinforcement. The failure of specimen 1C occurred in the welds between the headed anchor and the distribution plate. The ultimate loads determined from tests \( (N_{\text{test}}) \) are compared with the resistances according to CEN/TS 1992-4-2.1.
Figure 6 indicate that tensile resistances of headed anchors with supplementary reinforcement (1C, 2C, 3A) may be 3-3.6 times higher than resistances determined in accordance with the principles of current design method.

5 Conclusion

The executed experimental investigations have demonstrated that the resistance of headed anchors strongly depends on the presence and amount of longitudinal and shear reinforcement of concrete member. Therefore, the stiffness of the concrete member is a significant factor affecting the resistance and load-displacement behaviour of anchorage. According to test results obtained from test series C, it may be concluded that appropriate design and detailing of the supplementary reinforcement increases the ductility and resistance of the anchorage. The results of tests were compared with the values of resistance determined in accordance with the principles of CEN/TS 1992-4-2. It indicates that the technical specification considers the contribution of supplementary reinforcement too conservatively for the tested configuration of headed anchors with supplementary reinforcement.

References:


TENSION TESTS OF HEADED STUD ANCHORAGES IN NARROW / THIN EDGE MEMBERS

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ABSTRACT

The Precast/Prestressed Concrete Institute (PCI) sponsored a comprehensive research program to assess the capacity of headed-stud group anchorages. The PCI program was initiated in response to significant differences between headed stud design, as outlined in the PCI Design Handbook and the ACI 318 Building Code. Not covered by the PCI sponsored testing, but nonetheless important, were headed-stud groups located in thin or narrow members; that is, anchors embedded in a narrow member with two parallel free edges reducing the concrete breakout cone area. Tests of headed stud anchorage groups loaded in tension in this fashion are not extensively reported in the literature.

The test program herein, conducted by staff of Wiss, Janney, Elstner Associates, Inc. (WJE), was an in-house, unsponsored test program of 16 tension tests. Considered were the variables of embedment depth, edge distance, and spacing of the anchors parallel to the free edge. Test anchorages used one, two, and four headed studs in the connection. Transverse stud spacing, perpendicular to the edges, was also a secondary variable in this study. The results were compared to the present ACI 318-141, Chapter 17 provisions where two sides of the breakout surface are truncated. This paper provides a summary of the research work.

1 Introduction

Designing anchorages located in a narrow edge of a member are often avoided because of their perceived lower capacity attributed to the truncated concrete breakout cone from the small (narrow), two parallel side-edge distances. In ACI 318, Chapter 17 terminology, the edge distance from the anchor to the free edge is represented by \( c_{af} \) or \( c_{a2} \).

Placing an anchorage plate in the narrow edge of a floor slab to facilitate connecting a curtain-wall system is one typical example where this type of anchorage frequently occurs. Although this type of slab edge connection frequently carries gravity shear load, wind suction can also produce tension on these anchor groups. Conventional floor slabs range in thickness from 150 to 250 mm (6 to 10 in.), thus providing side-edge distances less than 75 to 125 mm (3 to 5 in.) if one line of anchors is used at mid-depth of the slab.
Research into the capacity of headed-stud anchorages located in a narrow edge and loaded in “pure” tension is limited. ACI 318, Chapter 17 provides a design method to handle this condition, but the design model appears to be extrapolated from the single-edge case. Experimental verification of this two-edge design model is limited. The ACI 318, Chapter 17 model accommodates this condition rather easily in the calculation of the $A_{Nc}$ term for concrete breakout area. In the long direction of the slab, the full $1.5 \ h_{ef}$ model breakout distances can be developed on both sides of the connection, provided that the anchorage is not near a corner. In the slab thickness direction, however, the breakout surface can be severely truncated by the actual side edges, $c_{a2}$, being less than $1.5 \ h_{ef}$. Moreover, the $\psi_{ed, N}$ term would be invoked to further reduce the capacity.

The authors had a unique opportunity to experimentally verify the tension capacity of this anchorage type located in a slab edge. Concrete slabs having dimensions of $1320 \times 1320 \ mm$ by $190 \ mm$ thick (4 ft - 4 in. x 4 ft - 4 in. x 0 ft - 7½ in.) were cast for another experimental project studying sealers. The edges of these four concrete slabs were used for the anchorage experiments reported herein.

### 2 Literature Review

The international tension database contains numerous tests on anchors and anchor groups with one edge of the concrete breakout cone truncated by the proximity of the edge. There do not appear to be any two-edge group tests reported in the tension database to the authors’ knowledge. Code design rules for conditions having two parallel free edges have been, in the authors’ opinion, extrapolated from the single-edge data to apply to cases where an anchorage is located in a slab edge that truncates the breakout cone on two opposite faces.

### 3 Experimental Program

#### 3.1 Test Slabs and Layout

Four test slabs were used. One anchorage was located on each side of the slab and centered in the length. Figure 1 shows a plan and section view of a typical anchorage placed in the edge of the slab. The slabs were cast flat so there was no “top bar” or casting position effect on the headed stud anchorages. A nominal amount of reinforcement was placed in the slabs for handling purposes. As shown in Figure 2, the reinforcement was placed below the anchorages to prevent inference with the breakout surface. General details of the 16 tests conducted are listed in Table 1.

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<th>Connection &amp; number of anchors</th>
<th>Number of tests</th>
<th>Stud dimensions</th>
<th>$c_{a1}$ (mm)</th>
<th>$c_{a2}$ (mm)</th>
<th>$s_1$ (mm)</th>
<th>$s_2$ (mm)</th>
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<td>5.30</td>
<td>616</td>
<td>66.7</td>
<td>0</td>
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<td>(2x2)</td>
<td>4*</td>
<td>12.5</td>
<td>5.30</td>
<td>616</td>
<td>66.7</td>
<td>88.9</td>
</tr>
</tbody>
</table>

where, $c_{a2}$ and $s_2$ are perpendicular to the free edge, as shown in Figure 1

*Note: One test in set damaged due to handling
As shown in Table 1, anchorages having one, two, and four headed studs were tested. The single-stud tests examined 15.8 mm (⅝ in.) diameter studs with a 94 mm (3.7 in.) embedment depth, and 12.5 mm (½ in.) diameter studs with a deeper 124 mm (4⅞ in.) embedment. The two-stud pattern with \( s_2 = 57.2 \text{ mm (2¼ in.)} \) was centered at mid depth and was replicated as one of the four-stud patterns with an \( s_1 \) spacing of 88.9 mm (3½ in.) or \( 1.33 h_{ef} \). The second set of four studs used a slightly larger spacing perpendicular to the free edge, \( s_2 \), of 79.9 mm (3⅛ in.), while the spacing parallel to the free edge was 81 mm (3⅛ in.) or \( 1.2 h_{ef} \). As addressed earlier, the slabs were used for another study. The slabs were over 1½-years old prior to testing, so the concrete properties had stabilized.

### 3.2 Concrete Properties

The concrete mixture design was similar to mixtures commonly used in precast construction. The concrete was a normal weight concrete weighing nominally 2320 kg/m\(^3\) (145 lbs/cu. ft or pcf). Mixture proportions and measured mechanical properties are as follows:
• 314 kg/m³ (530 lbs/cu. yd.) of Type I cement,
• 1094 kg/m³ (1844 lbs/cu. yd.) limestone coarse aggregate with a 19 mm (¾-in.) nominal maximum aggregate size,
• 825 kg/m³ (1390 lbs/cu. yd.) sand,
• water-cement ratio = 0.42, water reducer and air entertaining admixture included,
• measured slump of 172 mm (6¾ in.) and 7.8 percent air content,
• average compressive strength at 28 days: 30 MPa (4,440 psi),
• average compressive strength at time of anchor testing: 34.9 MPa (5060 psi),
• average splitting tensile strength at time of anchor testing: 3.1 MPa (455 psi).

3.3 Headed Stud Properties

The ultimate tensile strength of the headed studs varied from a low of 536 MPa (77.7 ksi) to a high of 563 MPa (81.6 ksi). All the load-deformation curves for studs tested in direct tension in air exhibited rounded, but ductile behavior. The ultimate tensile strengths of the studs used in the various anchorages are listed in Table 2.

3.4 Test Procedures

A fabricated steel self-equilibrating testing framework was used to conduct the tension tests in the slab edge and is shown in Figure 3. Each slab was tested flat, and the slab was rotated to access the embedded anchorage located in the center of the side. The test frame reacted near the ends of the slab edge, as to be located outside the potential breakout cone that forms along the long-edge direction of the test specimen. Test load was applied with a manually operated hydraulic ram, and load was measured with a load cell. Displacement instrumentation, shown in Figure 4, was measured continuously during the test to failure. Each test to failure took approximately 10 minutes to conduct. After the failure, the characteristics of all concrete breakout surfaces were documented.

Figure 3: Overall view of tension testing framework
3.5 Load-Deflection Behavior

Plots of the typical load-deflection behavior of a test sample with a steel failure and concrete breakout failures are shown in Figure 5. Two replicates were conducted of the single anchor configuration, while four replicates of each group test configuration were conducted. The steel failure plotted in Figure 5 is very ductile, exhibiting a shape similar to the stress-strain curve for the steel shank of the headed stud. This steel failure occurred with an embedment depth ratio, \( \frac{h_{ef}}{d_s} \), of 9.8 and edge distance to embedment depth ratio, \( \frac{c_a}{2h_{ef}} \), of 0.72. The other three curves are concrete failures and, as expected, are brittle in nature. Shown in Figure 5 are typical breakout behaviors of 1, 2, and 4 anchor groups.

![Figure 5: Load deformation behavior of anchors embedded in a slab edge](image)
3.6 Test Behavior

Table 2 presents the numerical test results, along with the actual measured values of the geometry and headed-stud properties. Observations of the failure surfaces follow:

Single Stud – Due to previous cracking damage, one of the 15.8 mm (5/8 in.) diameter stud tests was damaged. The other 15.8 mm diameter test exhibited concrete breakout. Along the long-edge direction of the concrete specimen, the typical conically shaped, breakout cone developed. In the narrow, thickness direction, the failure crack propagated horizontally to the two parallel edges; the breakout surface was essentially a flat without any sloping, conical, shape. This behavior was anticipated, as the slab thickness was slightly greater than $2h_{ef}$ for this anchorage. All the concrete breakout surfaces appeared like that shown in Figure 6.

Both 12.5 mm (1/2 in.) diameter single stud tests failed as steel failures. Steel rupture of the headed-stud shank occurred below the surface of the concrete and had a typical cup and cone failure surface. Figure 7 shows the failure.

Two Stud Group – All four tests exhibited concrete breakout. Full-cone development occurred in the long direction, whereas horizontal propagation of the failure surface crack extended to the two parallel edges, shown in Figure 8. The crack was nearly flat in the thickness direction and occurred at the level corresponding to the top of the head on the stud. All failure loads were reasonably consistent for the four replicate tests.

![Figure 6: Single stud concrete failure](image1)

![Figure 7: Steel failure, 12.5 mm stud](image2)
Four Stud Group – Figure 9 shows representative four-stud anchorage breakouts. The four-stud tests had a breakout behavior similar to the two-stud anchorage. The four-stud anchorage with an $s_1 = 57.2$ mm (2¾ in.) had an average failure load approximately 50 percent greater than the two-stud anchorage with the same stud spacing, $s_2$.

A comparison of the two sets of four-stud anchorages reveals an interesting behavior. The nominal stud spacing in the $s_1$ direction of the slab (parallel to the free edge) varied slightly, from $s_1 = 81.0$ to 88.9 mm ($3\frac{3}{16}$ to $3\frac{1}{2}$ in). However, the nominal stud spacing perpendicular to the narrow edge, $s_2$, varied from 57.2 to 79.9 mm (2¾ to 3¾ in.). Similarly, the edge distance to the narrow edge varied inversely. The four-stud connection with a wider $s_2$ spacing and smaller $c_{a2}$ edge distance had an average failure load about 13 percent greater than the smaller $s_2$ spacing, larger $c_{a2}$ edge distance combination. These results seem to indicate that the dimensions perpendicular to the parallel edge, or parallel to the thickness, may be the critical parameters associated with this slab edge type anchorage.
4 Analysis of Test Results

4.1 Steel Failure
For the two, single 12.5 mm stud tests failing in steel tension, the strength predicted using the known steel ultimate strength showed good agreement with the experimental results. The calculated strength under predicted the capacity by about 8 percent.

4.2 Concrete Breakout Failure
The tension breakout provisions of ACI 318-14, Chapter 17 were used to analyze the data, except the average basic breakout equation was used instead of the 5-percent fractile equation. Thus, the average basic breakout equation is:

\[ N_b = k_c \lambda_c \sqrt{f'c \cdot hef}^{1.5} \]  
\[ N_{cbg} = (A_{Nc}/A_{Nco}) \Psi_{nc,N} \Psi_{ed,N} \Psi_{c,N} \Psi_{cp,N} N_b \]

where:

- \( N_b \) = Nominal concrete tensile breakout strength - single anchor (N)
- \( N_{cbg} \) = Nominal concrete tensile breakout strength - group (N)
- \( k_c \) = 16.67 for the average value for cast-in-place anchors
  = 10.0 for the 5-percent fractile value for cast-in-place anchors
- \( \lambda_a \) = Lightweight concrete factor = 1.0 (for this test program)
- \( f_c' \) = Concrete compression strength measured on cylinders (MPa)
- \( hef \) = Effective embedment depth (mm)
- \( A_{Nc} \) = Actual projected area at the concrete surface (mm²)
- \( A_{Nco} \) = 9 \( hef^2 \) = single anchor breakout area (mm²)
- \( \Psi_{nc,N} \) = Eccentricity factor = 1.0
- \( \Psi_{ed,N} \) = Edge distance factor = 0.7 + 0.3(c_{a2}/1.5hef)
- \( \Psi_{c,N} \) = Cracked concrete factor = 1.0
- \( \Psi_{cp,N} \) = Post-installed anchor factor = 1.0

For the single, 15.8 mm stud anchorage, the ACI 318-14, Chapter 17 equation accounting for the parallel edges, \( c_{a2} \), under predicted the concrete capacity by 33 percent; the breakout prediction was conservative, and would be further conservative using the 5-percent fractile form of the equation, along with the appropriate safety factors. For the two-stud anchorage, the capacity was under predicted by 22 percent. The four-stud anchorages had under predicted capacities of 25 percent for one group and 52 percent for the other group. In all instances, the \( \Psi_{ed,N} \) factor was applied once, serving to reduce the predicted capacity.

When three or more edges exist within a connection, ACI 318-14, Chapter 17 provides for a reduced, effective embedment depth \( hef' \). Given the two-parallel edge conditions tested in this program, the reduced effective embedment depth was not considered, as it would have further reduced a conservative prediction. Thus, it was not applied to the two parallel edge conditions of these tests.
The above review of the test behavior indicated that the side-edge distance, $c_{a2}$, and anchor spacing parallel to the thickness, $s_2$, were important parameters in defining capacity. The two- and four-stud anchorages used a constant stud length, so the contribution of $h_{ef}$ to the capacity calculation was not studied. However, the embedment depth, $h_{ef}$, would only affect the concrete breakout surface projection along the long edge of the slab. In the thickness direction, the breakout will always be the horizontal thickness of the narrow edge, because the $c_{a2}/h_{ef}$ ratio is small. Thus the embedment depth does not affect the breakout in this direction.

Obviously, there will be a critical slab-edge thickness where the concrete breakout surface does not break through the slab thickness, and a cone will develop in the narrow, thickness edge dimension of the slab. This dimension will be dependent on the anchorage configuration and anchor embedment depth, and it may be $3h_{ef}$ or greater. Thus, there will be no constant thickness of a “thin” member that constitutes this behavior type.

Given the two- and four-stud anchorages of this test program, the test-to-predicted capacity was evaluated and plotted against the ratio of $s_2/c_{a2}$. This plot is presented as Figure 10. As shown in the plot, the test-to-predicted ratio increases as the $s_2/c_{a2}$ of the anchorage increases. The best-fit linear relationship is shown in the plot; for the 11 tests, the correlation coefficient, $R^2$, is 0.62. To provide a simpler factor that passes through 1.0, a parallel edge (||) modification factor for tension on anchors with two-parallel side edges is proposed as Equation 3:

$$
\Psi_{||,N} = 1.0 + (s_2 / 4c_{a2})
$$

Additional research is required beyond these 11 multiple-anchor tests to refine this factor. Alternately, perhaps the present edge distance modification factor, $\Psi_{ed,N}$, should be excluded from the group anchorage capacity, as it may not be entirely applicable to this anchorage configuration in a narrow edge. The present edge distance factor may be more applicable to a single edge. If excluded, the constant in the denominator in the proposed factor above should be doubled; that is, the 4 factor should be 8. Further study of narrow edges with anchorages in tension is warranted.

5 Conclusion

The present ACI 318-14, Chapter 17 provisions for tension, concrete breakout capacity are conservative when applied to a tension-loaded anchorage in a narrow member with two parallel edges. In the tests reported herein, the concrete breakout surface always broke through the narrow member thickness, such that the two dimensions of the failure surface perpendicular to the narrow edge dimension were truncated when making a calculation of capacity. For these tests, member thickness was always the default dimension in the breakout area computations. Given the 11 multiple stud anchorage tests, a modification factor for two parallel edges is proposed to increase the tension prediction using the CCD equations in ACI 318-14, Chapter 17, so it better matches the actual test behavior. This factor is derived from the testing reported herein and additional research into this configuration with different (various) edge thicknesses should be considered.
Figure 10: Test-to-ACI predicted capacity versus the spacing-to-edge distance ratio

6 Acknowledgement

The authors wish to express their appreciation to the Wiss, Janney, Elstner Associates, Inc. (WJE) for the use of their testing equipment and to Dr. John Lawler of WJE, who allowed the authors to cast these anchorages into the concrete slab edges for slabs being made for another purpose. The authors are also indebted the Precast/Prestressed Concrete Institute for initially sponsoring an extensive research project, which had a few leftover anchorages because the test program scope changed as information on the shear behavior of welded headed stud anchors became available.

References

1. ACI Committee 318 (2014), Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14), American Concrete Institute, Farmington Hills, Michigan, 519 pp.

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Notes:
- Column (1): TPET = (No. of Studs) – (Stud Diameter) – ($h_{ef}/d_a$ ratio) – ($s_2$ spacing) – (A, B, C, or D test) [units: inches]
- TPET = Two Parallel Edge Tests
- Column (14): † Test-to-predicted capacity computed with the steel strength
- Concrete cylinder strength = 34.9 MPa (5,060 psi)
- Conversions: 1 in. = 25.4 mm, 1 ksi = 6.895 MPa, 1 lb = 4.448 N, 1 kip = 4.448 kN, 1 lb/ft³ = 16.03 kg/m³
DESIGN OF THE ANCHORAGE OF INSERTS FOR LIFTING AND HANDLING OF PRECAST CONCRETE ELEMENTS

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ABSTRACT

Precast concrete construction allows a fast, economic and high-quality construction of structural buildings. Besides, inserts and insert systems for rapid and safe lifting and handling of concrete components have proven as essential in transport and erection. Nevertheless, up to now the regulations covering the use and the design of these fasteners are primarily based on experience as well as the results from product specific and application oriented tests. Hence, these regulations are not uniform in Europe, although, concerning the anchorage of the inserts, in the precast components the same physical conditions are given.

Inserts for lifting and handling are installed permanently in reinforced concrete precast components. However, they are used only temporarily and do not contribute to the mechanical resistance and stability of the completed structure. Their designated use ends with the assembly of the precast concrete components. For this reason inserts and insert systems for lifting and handling are not specified as construction products. In the sense of European regulations they are considered as lifting accessories, i.e. as a component of the complete load resisting chain consisting of crane, crane rope, crane hook, lifting key and cast-in lifting insert or lifting insert system consisting of lifting key and lifting insert. Thus they are covered by the European Machinery Directive 2006/42/EC1 and not taken care of under the umbrella of the European Construction Product Regulation2. The consequences for the design of the anchorage of the inserts according to resulting from this classification and the corresponding design rules of VDI/BV-BS 62053 and CEN/TR 157283 are discussed in greater detail.

1 Introduction

The demand for rapid high quality construction has resulted in an increased use of precast concrete elements. These structural components are moved from the precast plant to the construction site usually by means of lifting inserts or lifting insert systems. Lifting devices must function reliably. For this purpose they shall safely carry all actions resulting from transport, lifting operations as well as installation and transfer the loads to the structural component. The whole lifting system as it is considered by European regulations is shown in Figure 1.
A failure of lifting inserts and lifting insert systems could cause risk to human live and lead to significant economic loss. Therefore lifting inserts and lifting insert systems must be produced with high quality by the manufacturer, carefully selected and designed for the respective application by the designer and properly inserted and used by skilled personnel according to lifting and handling instructions.

In the following lifting inserts and lifting insert systems, their design according to actual design provisions, legal aspects and corresponding background information are presented.

2  Lifting inserts

The precast industry predominantly uses lifting inserts which belong to a serially produced lifting insert system consisting of a lifting insert and the corresponding lifting key (Figure 2). The lifting insert cast in the concrete component is used only a few times within the transport and assembling process of the precast element. After its use it remains in the concrete precast element. The corresponding lifting key which is normally linked to a specific manufacturer and lifting insert system is repeatedly used for further lifting and handling operations of other precast elements where the same type of lifting insert is installed.

Lifting inserts and lifting keys of different manufacturers are usually not compatible. Often lifting keys of one manufacturer can be screwed in the thread of the lifting insert of another manufacturer. This does not necessarily mean that the connection is sufficiently load-bearing, because the lifting insert can show, e.g. a round thread, while the lifting key has a sharp thread and both parts can be screwed together. Besides, round threads are not standardized, so that also a fit of round threads of different manufacturers normally cannot be considered as sufficiently load-bearing. Such incompatibilities between lifting inserts and lifting keys can yield the failure of the connection during
the lifting and handling operations with considerable health and economic consequences. This situation is dissatisfactory for everybody involved in the operation of precast elements. Hence, to avoid hazards due to incompatibilities it is highly desirable that lifting insert systems are modified in such a way, that

- not properly fitting parts cannot be connected, or
- failure occurs before the precast element is lifted,

or as the best alternative, that

lifting inserts and lifting insert systems are designed and marketed such that in any case an unambiguous assignment of compatible lifting inserts and lifting keys is simply perceivable and distinctive. The compatibility should cover products from different manufacturers and be stated in the corresponding lifting and handling instructions.

A system for lifting and handling of precast elements is normally limited to a specific range of application for example lifting of planar (e.g. slabs) or bar-shaped components (e.g. columns). This restriction ensures a safe and economic lifting and handling process. The specific ranges of application for the respective type of inserts are to be taken from the lifting and handling instructions of the manufacturers. Besides, these include the corresponding load-carrying capacities for a minimum concrete compressive strength of 15 N/mm² of the precast concrete element at the time of the first lifting operation and the minimum concrete component dimensions required for their application.

3 Design provisions

3.1 General

The use of lifting inserts and lifting insert systems is covered by the European Machinery Directive 2006/42/EC. It regulates only the steel parts of the insert i.e. the lifting insert exclusively as a product. Regulations for the concrete serving as anchorage material are not given. The load-carrying capacity of lifting inserts, however, is in most cases governed by concrete failure i.e. in the design the verification of concrete failure becomes decisive. Thus the properties and dimensions of
the precast concrete element serving as base for the anchorage of the inserts are essential – but they are not regulated in the Machinery Directive. This means, though lifting inserts are regulated in the area of harmonized standards, they are incompletely covered. Hence, essential aspects for closing this gap and to ensure the safe use of lifting inserts are included in different national and complementary documents.

EN 1992-4, Section 1.1. refers to the provisions of CEN/TR 15728 ‘Design and Use of Inserts for Lifting and Handling of Precast Concrete Elements’\(^3\) prepared by CEN/TC 229 for the design of the anchorage of lifting inserts in concrete. Nevertheless, in the present bulletin it is recommended to better use the VDI/BV-BS 6205 guideline ‘Lifting Inserts and Lifting Insert Systems for Precast Concrete Elements, Principles, Design, Applications’\(^4\), instead. Background to this recommendation is given in the following.

### 3.2 CEN/TR 15728

CEN/TC 229 ‘Precast Concrete Products’ wanted to create a base for the uniform and safe use and design of lifting inserts on European level with the first version of the Technical Report CEN/TR 15728 ‘Designs and Use of Inserts for Lifting and Handling’, published in 2008. This was an urgent need, because for comparable lifting inserts under comparable application conditions depending on the country and manufacturer partly contradictory design recommendations were given. In the meantime, the version of 2008 was basically reworked and all still existing inconsistencies were removed. The revised version was published in 2016.

CEN/TR 15728 applies exclusively to lifting inserts which serve the lifting and handling operations of precast concrete components produced of normal concrete. The production of the concrete components must be subject to a quality assurance process which includes the installation and the use of the lifting inserts.

The revised CEN/TR 15728 defines clearly and exemplarily the actual state of knowledge in the selection of lifting inserts as well as the test-supported determination of their characteristic and admissible resistances by means of clear requirements to the planning, performance and evaluation of tests. The safety concept is based on EN 1990.

The selection of the lifting insert according to CEN/TR 15728 occurs on the basis of the geometry of the precast component to be moved. It is distinguished between the applications in flat components as for example to walls or slabs, bar-shaped components as for example columns or beams, and massive components as well as pipes.

The first version of CEN/TR 15728 dating from 2008 contains models for the calculation and diagrams for the determination of resistances for tension loaded lifting inserts from serial production as well as manufactured in the precast plant for certain applications. These calculation methods were developed in the essentials from the technical information of catalogues of a very limited number of inserts and insert producers and therefore apply only for the suitable products. Nevertheless, these are not given in CEN/TR 15728, explicitly. Based on this fact and other weak spots which could imply a safety risk, this first version of CEN/TR 15728 was not translated into German and also not published in Germany.
Hence, it became necessary for Germany to develop the VDI/BV-BS 6205 guideline ‘Lifting Inserts and Lifting Insert Systems for Precast Concrete Elements’ which was released in 2012. It is much more detailed than CEN/TR 15728 and takes into account comprehensively the requirements of practice. The contents are discussed in the next section.

In the meantime, in parallel with the development of the VDI/BV-BS 6205 guideline on European level the intense work on a revision of CEN/TR 15728 was started. Essential knowledge from the VDI/BV-BS 6205 guideline was considered in the actual revised version of CEN/TR 15728 from 2016. In particular the section including the design of lifting inserts, which at present contains the EN 1992-4 model as a basis for the design, was fundamentally revised. Now a sufficient safety level is ensured during lifting and handling of precast concrete products with lifting inserts.

For the sake of completeness it has to be mentioned that CEN/TR 15728 is a state of art report in the area of structural engineering. For reasons of the unclear legal situation – a document provided under the umbrella of the Construction Products Regulation covering a product covered by the Machinery Directive - CEN/TR 15728 excludes its use as an interpretation paper to the European Machinery Directive in 2006/42/EC which regulates the uniform and practical use of lifting inserts and lifting insert systems as load accessory in Europe.

3.3 VDI/BV-BS 6205

The German precast industry uses serially produced lifting insert systems for lifting and handling operations of structural precast components in most applications. Hence, the VDI/BV-BS 6205 guideline ‘Lifting Inserts and Lifting Insert Systems for Precast Concrete Elements’ focusses on this type of products. In the special case where a precast factory produces its own special lifting inserts it is highly recommended to analogously use and follow the VDI/BV-BS 6205 rules to create a comparable safety level.

With the publication of the VDI/BV-BS 6205 guideline for the first time a uniform and comprehensive procedure was presented for the design, the construction as well as the determination of the resistance and use of lifting inserts. Besides, the basic requirements of the Machinery Directive as well as the parameters relevant for practice as shown in Figure 3 were considered.

The lifting insert producers are given default values, e.g. concerning the ductility of the steel to be used in production. Thus it is ensured that in case of appropriate use even at very deep temperatures a big deformation capacity exists so that the precast element can be safely handled.

The application oriented testing of lifting inserts and the corresponding evaluation procedure conform substantially to the rules of CEN/TR 15728 and permit the determination of reliable resistances appropriate for the technical documents of the lifting inserts e.g. the lifting and handling instructions.

The contents of the lifting and handling instructions which the producer of the lifting insert must make available to the user for the design and the installation of the lifting insert as well as for the handling of the precast element are also defined. These specifications have to be observed by the user to ensure the safe use of the selected lifting inserts.
The VDI/BV-BS 6205 guideline ‘Lifting Inserts and Lifting Insert Systems for Precast Concrete Elements’ is the basis for the proper use of lifting inserts, is application oriented and consists of 3 parts (Figure 4):

– Part 1: General Principles
– Part 2: Manufacturing and Placing on the Market
– Part 3: Design and Application

Part 1 incorporates general principles to be considered by producers, designers and users. Part 2 addresses producers and suppliers of lifting inserts and the focus of Part 3 are designers and users. The principles and requirements of Part 2 and Part 3 are supplementary to Part 1. This means that designers and users need Part 1 and Part 3 for the proper use of lifting inserts, and producers Part 1 and Part 2 for the professional production of lifting inserts and the development of the corresponding lifting and handling instructions.

Deviating from the usual approach of the Eurocodes and EN 1992-4 the safety concept corresponds to the demands of the Machinery Directive. Hence, the verifications of the load-carrying capacity are not performed at the design level, but on the basis of the admissible load level by means of global safety factors.

\[ E \leq R_{\text{adm}} \]  

with:

- \( E \): load acting on a lifting insert
- \( R_{\text{adm}} \): admissible load (resistance) of a lifting insert
\[ R_{adm} = \frac{R_k}{\gamma} \]  \hspace{1cm} (2)

with:

- \( R_k \): characteristic value of the resistance of the anchorage of a lifting insert pursuant the installation and lifting instruction, depending on the selected lifting insert and its application
- \( \gamma \): global safety factor, dependent on the failure mode, factor to cover uncertainties in action and resistance, after VDI/BV-BS 6205 and Machinery Directive in case of steel failure, respectively

The global safety factors in case of steel failure of lifting inserts produced from steel ropes amount to \( \gamma = 4.0 \), for lifting inserts made from chains to \( \gamma = 3.0 \) and for lifting inserts produced from full cross sections to \( \gamma = 3.0 \). For the failure modes concrete break-out, splitting, blow-out and pull-out the global safety factor depends on the testing and evaluation procedure and is \( \gamma = 2.5 \) or \( \gamma = 3.0 \). If lifting inserts are installed in precast elements under plant specific and continuous inspection the global safety factor may be reduced to \( \gamma = 2.1 \) or 2.5.

![Figure 4: Interaction between the different parts of VDI/BV-BS 6205](image)

Part 2 ‘Manufacturing and Placing on the Market’ is concerned exclusively with aspects relevant for the producer which are based on the demands of the Machinery Directive in 2006/42/EC. They begin with the design criteria for lifting inserts, contain rules for the verification of the suitability and load-carrying capacity for specific applications and end with definitions with respect to the documentation of technical data in the lifting and handling instructions. Special attention is given to the process for the exclusion of significant hazards and the avoidance of foreseeable misuse.
The determination of the resistance of lifting inserts can occur according to Part 2 on the basis of tests, proof loading and physically based design models.

The rules for the performance of tests and the evaluation of the test results agree in principle with the recommendations given in CEN/TR 15728. Hence, the test and evaluation methods for lifting inserts and lifting insert systems are clearly defined and ensure reproducible values as a basis for the design on a uniform safety level. A contemporary semi-probabilistic safety concept is used.

The conformity of lifting inserts and the corresponding technical information with the demands of the VDI/BV-BS 6205 guideline leads to high-quality products. Hence, the manufacturer is permitted to highlight the conformity of his product with e.g. the addition of the term ‘VDI/BV-BS-6205-konform’ in the installation and handling instructions of the lifting inserts.

Designers and precast plant staff find all relevant information for the correct selection and design of the lifting inserts in VDI/BV-BS 6205, Part 3 ‘Design and Application’.

Part 3 contains rules for the verification of the serviceability limit state and the ultimate limit state of lifting inserts. Although lifting inserts are subjected only to transient actions, it is to be paid attention to the fact that they do not unfavorably influence the serviceability of the precast element during its service life, e.g., due to corrosion. For the verification of the load-carrying capacity instructions are given with respect to the static system to be selected for the distribution of the load to the single lifting inserts anchored in a precast component. The loads acting on the inserts are to be determined according to the different lifting and handling conditions taking into account the effects from the dead load of the precast element, form adhesion and friction during lifting from the formwork as well as dynamic actions depending on the hoist (type of crane, excavator…) and the condition of the terrain on which the precast element is transported and the governing load combinations. For the calculation of the resistance the technical documents of the lifting insert producer have to be observed. The resistance, however, can be also determined by means of proof loading at the corresponding precast component. The process of proof loading is described in detail in Part 2. The computational verification of the anchorage of lifting inserts can occur also on the basis of EN 1992-4 which is valid for the determination of the resistance to structural loads. Nevertheless, it shall be mentioned that in most cases for lifting inserts this method of verification yields very conservative results. This is due to the fact that lifting insert applications are intended for transient load transfer only, very specific and usually particular measures such as addition of special supplementary reinforcement are taken to allow for lifting of the precast element at an early age.

Part 3 covers also the practical application of lifting inserts and lifting insert systems during the lifting and handling operations of reinforced concrete and pre-stressed concrete precast elements produced from normal concrete. To avoid foreseeable misuse during the installation and lifting and handling directions to the necessary documentation are given, in addition. Hereby it is ensured, e.g. that a lifting key compatible to the lifting insert is used for lifting and handling of the precast element and that the precast element is stored according to the assumptions made in the lifting insert design.

The actual version of CEN/TR 15728 represents state of the art. VDI/BV-BS 6205 is much more comprehensive and detailed and not outdated. However, it is recommended to update VDI/BV-BS 6205 to be in complete agreement with CEN/TR 15728.
4 Conclusions

The anchorage of lifting inserts and lifting insert systems is not covered by the European standards and only insufficiently addressed by the Machinery Directive. Up to now there is no European supplement or interpretation document which closes the existing gaps.

The CEN/TR 15728 version from 2008 represents a first step in this direction, nevertheless, includes design models for the determination of the resistance of lifting inserts which can yield very liberal results. This deficiency is settled by the revised version of CEN/TR 15728 published in 2016. Besides, CEN/TR 15728 explicitly excludes its use as an interpretation document of the Machinery Directive. In Germany the normative gap is closed by the VDI/BV-BS 6205 guideline. It corresponds in the essentials with the revised CEN/TR 15728 and gives additional directions and explanations for the proper use of lifting inserts and lifting insert systems. Hence, it is recommended to follow EN 1992-4 and to use CEN/TR 15728 in the actual version and the guideline VDI/BV-BS 6205 as supplement.

References:


SUPPLEMENTARY REINFORCEMENT FOR CAST-IN FASTENERS

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ABSTRACT

Cast-in fasteners such as headed bolts and anchor channels are preinstalled during the construction of concrete or reinforced concrete structures. Such fasteners offer the possibility to either use the existing reinforcement for load transfer to or precisely define an additional reinforcement. This additional reinforcement - mentioned in EN 1992-4 as supplementary reinforcement - can transfer tensile or shear forces to another region of the component, depending on the position and orientation of the cast-in fastener.

According to EN 1992-4 the verification of the supplementary reinforcement shall be performed for the highest loaded anchor. The calculated amount of reinforcement shall be applied for all fasteners of the group. As an alternative, the verification may be done for the group. This is more economic for large fixtures with several rows of fasteners close to corners or neighboring fastener groups. In this paper, additional rules, which go beyond the regulations in EN 1992-4, will be provided. Furthermore, guidance for the rational and economical use in practice and construction rules for supplementary reinforcement used with cast-in fasteners will be discussed.

1 Introduction

Cast-in fasteners such as headed bolts and anchor channels are preinstalled during the construction of concrete or reinforced concrete structures. This means they offer the possibility to either use the existing reinforcement for load transfer or precisely define an additional reinforcement. This additional reinforcement - mentioned in EN 1992-4 as supplementary reinforcement - can transfer tensile or shear forces to another region of the component, depending on the position and orientation of the cast-in fastener.

In this paper, construction rules for additional reinforcements used with cast-in fasteners, which go beyond the regulations in EN 1992-4, will be discussed and guidance for the rational and economical use in practice will be provided.

2 Functioning principle and design format

The operating principle of the additional reinforcement is as follows: With fixings under tension load or close to an edge under shear load, the supplementary reinforcement shall tie the concrete breakout body to the concrete member. The reinforcement is recognized to be anchored in the idealized
breakout body. Thus a cracked state is assumed. The concrete can transfer only small amounts of the load in this state. The supplementary reinforcement must carry the entire tension or shear load. The verification of concrete failure (tension: concrete cone failure; shear: concrete edge failure) may then be omitted because with activation of the supplementary reinforcement the breakout body is already formed. The serviceability is considered via admissible displacements or crack widths. Fig. 1 shows a supplementary reinforcement for tension load and the corresponding strut-and-tie model.

\[ \pm 0.75 h_{cr} \]

Figure 1: Supplementary reinforcement for tension load (left) and the corresponding strut-and-tie model (right) \(^1\)

For determination of the characteristic resistance of the supplementary reinforcement two verifications must be performed:

- Anchorage failure of the reinforcement in the concrete breakout body, and
- Steel failure of each effective bar of the supplementary reinforcement.

The proof of the anchorage failure of the supplementary reinforcement corresponds to the rules for anchorage of rebars acc. to EN 1992-1-1\(^2\). The equation for calculating the required anchorage length \( l_1 \) was rearranged for the load in order to calculate the anchorage load of each individual rebar taking into account the existing anchorage length within the concrete breakout body (compare EN 1992-4). Only rebars within a certain distance from the fasteners (headed stud or anchors of anchor channels) are considered effective. The supplementary reinforcement should be adequately anchored outside the assumed breakout body with an anchorage length according to EN 1992-1-1.

Furthermore each single bar should not be loaded with a load higher than its tensile strength. Therefore the additional proof of steel failure is required.

The load-bearing capacity is strongly influenced by the rebar diameter (steel failure) and the anchorage length in the breakout body. This length is depending on the edge distance for shear loading and on the embedment depth of the fastener for tension loading. The larger the edge distance \( c \) or the embedment depth \( h_{ef} \), the greater the anchoring length of the individual reinforcement bars in the assumed breakout body and the more rods in width can be used for load transfer. Following this, a certain anchorage length exists, beyond which always steel failure occurs. A larger anchorage length is not appropriate.
For shear loads, an alternative for fastenings with small edge distance exists. Looped or hairpin-shaped reinforcing bars close to the concrete surface and in contact with the shaft of the fastener are inserted (see Fig. 2). This alternative is given in EN 1992-4, but not illustrated.

For shear loads, an alternative for fastenings with small edge distance exists. Looped or hairpin-shaped reinforcing bars close to the concrete surface and in contact with the shaft of the fastener are inserted (see Fig. 2).

In that case the shear load is not transferred via the breakout body to the reinforcement, but via direct compression contact between the loop or stirrup and the anchor shaft. This direct contact can hardly be guaranteed in practice and therefore the bearing capacity is limited. Due to a small gap between anchor shaft and reinforcement as well as due to inclinations of the stirrups/loops compared to the load direction the load-bearing capacity is limited to 50% of the tensile strength of the rebar. However, with this type of supplementary reinforcement the proof of the anchorage of the reinforcement in the breakout body does not apply. This is beneficial for fastenings close to the edge, since the existing small anchorage length may not be sufficient to anchor the applied load.

With anchor channels, the arrangement of loops as shown in Fig. 2 is less effective compared to headed studs. This is based on the fact that the loops must be positioned underneath the channel which results in a rather large distance between concrete surface and the loops. With the exception of T-headed anchors welded perpendicular to the channel axis, the stiffness of the connection between anchor and channel is rather low. The use of stirrups according to Fig. 3 is more reasonable.

Generally, a crosswise reinforcement close to the concrete surface is present, which can be used for the shear load transfer. This reinforcement must be able to carry these loads additionally to the existing loads from the structure (see Fig. 3, left). Therefore a closely spaced reinforcement arrangement is reasonable. Furthermore the eccentricity between axis of reinforcement and line of shear force shall be taken into account (see Fig. 3, right).

For combined tension and shear loads, a rather conservative approach for the $N - V$ Interaction is required in EN 1992-4, if supplementary reinforcement for tension or shear load only is present.

With fixings with supplementary reinforcement in either tension or shear direction, pronounced breakout bodies appear well before reaching maximum load capacity. These breakout bodies can affect the other load direction unfavourable. Until now, these conditions have not been investigated.
in detail. Therefore, rather conservative approaches for headed studs and anchor channels must be used. For headed studs, an increase of the embedment depth is more beneficial than arranging supplementary reinforcement (compare section 4).

Fig. 3: Hanger reinforcement for shear load with corresponding strut-and-tie-model (left) and calculation of the residual tension force in the reinforcement due to eccentricity between axis of reinforcement and line of shear force (right)\(^1\)

### 3 Recommendations for the reinforcement arrangement

Generally, hanger or supplementary reinforcement for fixings loaded in tension is placed additionally to the existing reinforcement. C- or Z-shaped hooks as well as stirrups (U-shaped or closed) can be used.

For hooks and loops the minimum mandrel diameter \(D\) according to DIN EN 1992-1-1\(^2\), section 8.3 may be used. For an increase of the bond strength a mandrel diameter \(D \geq 15\phi\) for a concrete cover larger than 3\(\phi\) may be used. On the loaded side of the concrete member, the supplementary reinforcement for tension load may enclose the bending reinforcement or not since the anchorage length must be checked anyway. For the second case, the resulting anchorage length is shorter.

Fig. 4: Possible positions of the supplementary reinforcement on the loaded side of the concrete member

For structural reasons and for easier installation, in general the supplementary reinforcement shall reach up to the unloaded side of the concrete member and enclose the existing reinforcement (compare hanger reinforcement acc. to EN 1992-1-1\(^2\), section 6.2.1 (9)). The anchorage length of
the supplementary reinforcement is for the frequent applications in walls and slabs limited by the thickness of the member. The anchorage outside of the idealised concrete breakout body should be detailed as follows. The existing reinforcement on the opposite side of the concrete member should be enclosed or the supplementary reinforcement bended parallel to the concrete surface with a large mandrel diameter to guarantee the required anchorage length $l_{b,net}$ (see Fig. 5).

![Fig. 5: Position of the supplementary reinforcement on the unloaded side of the concrete member with small member depth](image)

Supplementary reinforcement for shear loads should be positioned close the concrete surface to reduce the eccentricity between the axis of the reinforcement and the line of shear force (see Fig. 3, right). This is generally the fact in case of surface reinforcement used as supplementary reinforcement, because this reinforcement is placed usually in the first and second reinforcement layer. This is often not possible with additionally arranged loops according to Fig. 2. The loops are often placed in the third or fourth reinforcement layer (fastening in the corner of a concrete member).

4 When is a supplementary reinforcement useful?

Supplementary reinforcement for tensile loads is only useful if a large anchorage length of the supplementary reinforcement in the idealised concrete breakout body is present (see section 2). This is the case for large component thickness or if a relatively large length of the bolt is used. Then the lap length between bolts and supplementary reinforcement is sufficiently high. Furthermore, the load in the supplementary reinforcement can be relatively well-anchored and transferred into the concrete member.

For small member thickness, either the load outside of the breakout body cannot be anchored adequately or the lap length of the supplementary reinforcement and the bolts is so small, that the bond strength in the idealised concrete breakout body is decisive. Thus, only a portion of the available steel resistance of the bolt or of the supplementary reinforcement can be used.

Therefore with small member thickness it is often more effective to increase the anchorage depth of the headed bolts instead of arranging a supplementary reinforcement. This is possible by using correspondingly large supply lengths of the bolts or multiple welding of stacked single bolts.

With larger anchorage length, the load capacity increases - and a loss of performance due to the load transfer from the bolt via the concrete to the reinforcement does not apply. Ideally, the anchorage length is at least 80% of the component thickness ($h_{ef} \geq 0.8h$). Then, the anchoring of the tensile
loads in the compression zone of the concrete member is ensured (see also EN 1992-4, Annex A.2). Moreover, the unfavourable interaction according to EN 1992-4 can be avoided. A limit for this procedure is present for fastenings close to an edge and if an influence on neighbouring anchors shall be avoided. In these cases a supplementary reinforcement for tension load may be more appropriate.

Furthermore, supplementary tension reinforcement may be used for existing fixings. In case of load changes an existing reinforcement may be taken into account which was not planned as supplementary reinforcement in the original design but fulfils the requirements according to EN1992-4.

For supplementary reinforcement in shear direction, different types must be considered: surface (mesh) reinforcement and closely positioned loops.

In most cases, reinforced concrete elements contain crosswise reinforcement close to the concrete surface which can be used for fastenings under shear load close to an edge. This is especially valid for fastenings with an edge distance \( c_1 > h_{ef} \) and concrete elements with small member depth (which decreases the concrete edge resistance). This is often possible even for the recalculation of present components due to load changes and for post-installed fixings. Whether the existing reinforcement, and in particular the edge reinforcement can carry the additional forces, must be checked.

The second type of shear reinforcement in form of additional loops is generally planned and designed for the loads. This type of reinforcement is useful with fastenings close to an edge and concentrated, large loads in direction to the edge.

5 Further provisions for headed studs

5.1 Verification of the group

According to EN 1992-4 the verification of the supplementary reinforcement shall be performed for the highest loaded anchor. The calculated amount of reinforcement shall be applied for all fasteners of the group. As an alternative, the verification may be done for the group. This is more economic for large fixtures with several rows of fasteners close to corners or neighbouring fastener groups. Furthermore eccentric loads may be considered. The advantage is that the applied load may be distributed among several parallel oriented rebars. The theoretical background is based on the factors \( \psi_{ec,N} \) and \( \psi_{ec,V} \) taking into account eccentric load application within the verification for concrete cone and concrete edge failure. In general, the supplementary reinforcement retains the concrete break-out body as a whole. Therefore the external loads can be compared directly with the load-bearing capacity of the reinforcement. This will be shown with the following example consisting of a surface reinforcement used as supplementary reinforcement for shear loading.

Fig. 6 shows the ideal case of a fastener group without influencing lateral edges or neighbouring groups. The load is applied in the centre of the group; therefore no eccentricities between applied load and the reinforcement are present.
Fig. 6: Surface reinforcement used as supplementary reinforcement for shear loading – undisturbed area

Fig. 7 shows a fastener group close to a corner, where due to different anchorage lengths \( l_{1,i} \) the resultant of the different forces in the rebars is acting eccentric with regard to the centre line of the anchor plate and the group \( (\Sigma N_{Rd, re} = \Sigma_{1-i} \min (N_{Rd, re, i}; N_{Rd, a, i})) \). This results in an eccentricity \( e_{V,1} \). Furthermore the applied load is acting eccentric with regard to the centre line of the anchor plate, which causes a second eccentricity \( e_{V,2} \).

For the calculation of the load-bearing capacity of the supplementary reinforcement, the eccentricity \( e_V = e_{V,1} + e_{V,2} \) is used and the sum of resistances of the single rebars \( \Sigma N_{Rd, re} \) is reduced with the factor \( \psi_{ec,V} \).

5.2 Group of headed studs with multiple rows of fasteners

No provisions are given in EN 1992-4 for the verification of the supplementary reinforcement in shear direction for groups of headed studs with multiple rows of fasteners. Due to the absence of hole
clearance for headed studs used in fixings all anchors participate in the load transfer. Within groups with maximum three anchor rows in load direction it can be assumed that the applied shear load is distributed evenly among the anchors (see paper “Load distribution under shear load” in part 3).

This leads to the conclusion, that within the calculation of the load-bearing capacity of supplementary reinforcement for fixings consisting of multiple rows of anchors in load direction, different anchorage lengths $l_i$ for the rebars must be considered (see Fig. 8).

![Fig 8: Supplementary reinforcement for shear load with groups of headed studs with multiple anchor rows](image)

Following the assumptions in [4], section 10.2.5.1.1 for fixings consisting of multiple rows of anchors for concrete edge failure, two verification methods are possible:

a) General approach: The shear load is distributed to all participating anchors and the verification is done for the row of anchors closest to the edge. The anchorage length $l_i$ is depending on the edge distance of this row. This is a general approach, but conservative, since the longer anchorage length of the rows further away from the edge is neglected.

b) Special approach: One or more rows of anchors close to the edge are considered as failed, the verification is done at the next row further away from the edge. Only the anchors located far from the edge are considered as effective.

This assumption is only valid under the following conditions:

- The shear load is directed to the edge. Shear load inclined or parallel to the edge in combination with the rows of anchors considered as failed, generate an eccentricity between the point of load application and the centre of the load-bearing anchors. In this case, a torsion moment must be considered (see [4], Table 4.3-2).

- The verification of steel failure and concrete pry-out failure is performed only for the anchors far from the edge considered as effective.

The special approach leads to higher load-bearing capacities compared to the general approach in case of large anchor plates close to the edge and large shear loads.
6 Summary

For fastenings consisting of cast-in elements like headed studs and anchor channels the existing or additional reinforcement can be used for the load transfer. This supplementary reinforcement may be used for specific load transfer of tension and/or shear loads (depending on the position and orientation) to the concrete member.

This paper gives additional provisions to EN 1992-4 for cast-in fasteners in combination with supplementary reinforcement. Recommendations for the practical and efficient use of supplementary reinforcement are given.

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PULL-OUT CAPACITY OF CAST-IN HEADED ANCHORS IN PREFABRICATED CONCRETE ELEMENTS

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ABSTRACT

Headed anchors are used commonly as an anchoring system to lift prefabricated concrete elements. For thin concrete civil elements, such as concrete pipes, headed anchors with small embedment depth are typically used. An observed type of failure using a shallow embedded headed anchor is pull-out failure. This mode of failure is characterized by anchor pulling out a cylindrical shape of concrete, which occurs prior to the onset of concrete cone failure. The European and Australian guidelines to calculate the pull-out strength of headed anchors is reviewed in this paper and compared against experimental results and numerical simulation using program ATENA.

Experimental investigation on the load carrying capacity of cast-in headed inserts subjected to tensile loading is presented. Seventy two test specimens are analysed for failure mode and ultimate load. The crack width and crack propagation is reviewed and compared in the numerical modelling conducted in ATENA. The load was applied uni-axially, whilst the applied load, displacement and crack width was monitored. The parameters considered to initiate a pull-out failure mode, and used in the numerical simulation were (1) effective embedment of the headed inserts, (2) concrete compressive strength, (3) Poisson’s ratio, and, and (4) modulus of elasticity for each concrete mix type. This study demonstrates the application of fracture mechanics principles as the fundamental model used to predict failure of headed inserts for a pull-out failure mode. The experiment was simulated numerically by program ATENA. The experimental observations were analysed to compare the fracture surfaces against the numerical simulation.

The results of an experimental program, supported by numerical simulation, of pull-out strength of various embedment depths of headed inserts in different concrete strengths are presented. The accuracy of using numerical simulation is compared against published models to predict the pull-out failure capacity of inserts in early hydration age concrete.

1 Introduction

Cast-in headed inserts are a common type of anchorage system for lifting and movement of prefabricated concrete elements such as tilt-up panels, precast walls and civil elements. Headed
anchors are available in different sizes and lengths to suit their application. Various materials are used to create the void around the lifting lug of the anchor, and most typically rubber. The anchor does not sit proud of the concrete surface, as the void is often grouted after the final lift is completed. Further, these void formers are used to keep the anchor in place during the cast, by using a bolt and nut. Figure 1 shows the assembly of a headed anchor and void former before the cast.

![Figure 1: (a) and (b) Headed anchor assembled to the void former (c) and (d) the anchor is attached to the formwork before casting](image)

After casting, the lifting clutch is used to connect the lifting chain to the lifting point of the anchor. The clutch connects to the anchor by lowering the clutch slot over the anchor and rotating the clutch tab as shown in Figure 2 (b).

![Figure 2: (a) Lifting clutch is connected to the headed anchor (b) precast element is lifted and anchor undergoes tension loading](image)

The tensile strength of the headed anchor is determined considering the failure of the anchor steel or concrete surrounding the anchorage. Most internationally recognised guides for design of headed anchors such as AS3850.1, CEN/TS1992-4-2, ACI 318-11 and fib 4 provide concrete capacity models to calculate the failure of concrete in tension as concrete cone failure or the crushing of concrete around the foot of the headed anchor as shown in Figure 3.
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Figure 3: Pull-out failure and concrete cone failure considered in AS3850.1

For thin precast elements, such as concrete pipes, headed anchors with a small embedded length are used in the thickness of the element. The use of a void former combined with short headed anchors can result in a distinctive failure mode which has been overlooked in AS3850.1, CEN/TS1992-4-2, ACI 318-11 and fib Concrete Capacity Design models. The failure mode is described as Pull-out, caused by the anchor, and where the concrete cracks directly to the lowest part of the void from the anchorage. This is characterised by the anchor pulling out a cylinder of concrete, and which occurs prior to the onset of concrete cone failure, as shown in Figure 4.

Figure 4: Pull-out failure through the void former of a shallow embedded headed anchor

In this paper the tensile strength of shallow embedded anchors exhibiting a pull-out failure mode through the void former and concrete cone failure are investigated and recommendations are made...
for further research. The experimental results are compared to a numerical simulated model, using
ATENA, and the significance of different material parameters are compared.

2 Background

In Australia, lifting design engineers use either AS3850.1 Appendix B to calculate the tension
strength of headed anchors or use capacity tables from the anchor manufacturer. The capacity tables
are derived from a comprehensive testing programme which is outlined in AS3850.1 Appendix A.

Appendix A, AS3850.1, requires a test program using a reference anchor, Thjs ‘reference’ anchor is
to have a round body with minimum net bearing area of four times the cross-sectional area of the
body of the anchor, and the head diameter to be more than 2.4 times the shaft diameter. If a reference
anchor is positioned more than 1.5 times its effective embedment depth from edges, three possible
modes of failure are described to be associated with the reference cast-in headed anchor, namely
steel failure, concrete cone failure and pull-out failure.

AS3850.1 Appendix B sets out a design procedure similar to CEN/TS1992-4-2, which is based on
Concrete Capacity Design (CCD) method, explained5, 6. The basic characteristic resistance of a
reference cast-in headed anchor in case of concrete cone failure, N_{u,c}, is given as Eq.(1).

\[ N_{u,c} = \frac{k_{cr}}{f'_{c,age}} \cdot h_{ef}^{1.5} \] (1)

Where:  \( k_{cr} \) is 10 in the case of cracked concrete and 13 in the case of non-cracked concrete,
\( f'_{c,age} \) is the characteristic compressive concrete cylinder strength at the time of loading the
anchor, in MPa, and
\( h_{ef} \) is the effective embedment depth of the headed cast-in anchor in mm as shown in Figure
5.

The characteristic resistance of a reference headed anchor, AS3850.1 in case of pull-out failure is
given as Eq.(2).

\[ N_{u,p} = \frac{\pi}{4} \cdot (d_{i,h}^2 - d_i^2) \cdot f'_{c,age} \cdot \psi_{ucr,N} \] (2)

Where:  \( d_{i,h} \) is the diameter of the head of anchor,
\( d_i \) is the diameter of the shank of anchor as shown in Figure 5,
\( \psi_{ucr,N} \) is 1.0 for cracked concrete and 1.4 for non-cracked concrete.

Comparing Eq.(2) to the pull-out equations, given in CEN/TS1992-4-2 section 6.2.4 and fib4 section
19.1.1.3, a factor of 7.5 is missing in AS3850.1 clause B3.2.2.1 and the pull-out strength is
recommended to be expressed as Eq.(3).
\[ N_{u,p} = 7.5 \cdot \frac{\pi}{4} \cdot (d_{i,h}^2 - d_i^2) \cdot f'c,age \cdot \psi_{ucr,N} \] (3)

AS3850.1\(^1\) does not provide any design guide to check the capacity of shallow anchors for the case of pull-out failure of headed anchor through the void former, shown in Figure 4.

![Figure 5: Effective depth, head diameter and shank diameter of a reference headed anchor as described in AS3850.1\(^1\)](image)

### 3 Experimental results

A commercially available headed anchor, which satisfies the requirements for the reference anchor in AS3850.1\(^1\), was tested in non-cracked concrete with 50mm, 60mm and 70mm effective embedment depths. To investigate the effect of void former on the performance of anchors, two variations of anchor setup were used: first set of anchors were placed assembled with a 30mm radius void former, \( r_v \), and the second set of anchors were placed with similar effective depth but no void former as shown in Figure 6. Both sets of anchors were set to the same effective embedment depths.

The testing reaction frame included a 40mm thick steel ring with 340mm diameter, which was placed concentrically around the axis of each anchor, and a stiff beam sitting on the ring at four points. A loadcell and Linear Variable Displacement transducer, LVDT, were used to record the measurement of the applied tension load and displacement of the anchor following the requirements of AS3850.1\(^1\) clause A7.5 for testing of anchors in tension, refer Figure 7.

![Figure 6: Two variations of the anchor setup for testing in tension](image)

a) Test specimen cast with void former  b) Test specimen cast with no void former
Anchors were cast in a 300mm thick concrete block to avoid a flexural crack in the test blocks whilst the test load is applied. A 32MPa compressive design strength concrete mix was used throughout the experiment, using maximum 14mm course aggregate. Slump prior to casting was measured at 85mm. Anchors were tested in groups of eighteen at concrete hydration ages of 1 day, 2 days, 3 days and 7 days after casting. A sample size of three was used for three different effective depths i.e. 50mm, 60mm and 70mm in two variations of anchor setup shown in Figure 6. 72 anchors were tested in total, as per the test matrix denoted in Table 1.

Table 1: Test sample size of used for both voided and unvoided headed anchors

<table>
<thead>
<tr>
<th>Effective embedment depths, mm</th>
<th>Concrete hydration age, days</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Day 1</td>
</tr>
<tr>
<td>50mm</td>
<td>3</td>
</tr>
<tr>
<td>60mm</td>
<td>3</td>
</tr>
<tr>
<td>70mm</td>
<td>3</td>
</tr>
</tbody>
</table>

Three concrete compression cylinder samples were tested at each hydration interval of the experiment to determine the mean concrete compressive strength at time of test, poisson’s ratio and modulus of elasticity. Table 2 presents the mean values of concrete compressive strength, $f_{c,\text{age}}$, concrete modulus of elasticity, $E_c$ and poisson’s ratio, $\nu$. 
Table 2: Concrete properties at the time of testing

<table>
<thead>
<tr>
<th>Day after cast</th>
<th>$f_{c,age}$ (MPa)</th>
<th>$E_c$ (MPa)</th>
<th>$\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Day1</td>
<td>7.7</td>
<td>15000</td>
<td>0.24</td>
</tr>
<tr>
<td>Day2</td>
<td>13.0</td>
<td>21000</td>
<td>0.20</td>
</tr>
<tr>
<td>Day3</td>
<td>16.0</td>
<td>22000</td>
<td>0.21</td>
</tr>
<tr>
<td>Day7</td>
<td>21.0</td>
<td>26000</td>
<td>0.19</td>
</tr>
</tbody>
</table>

Table 3 compares the ratio of pull-out strength of the tested anchors, calculated by Eq.(3), over the concrete cone strength, calculated by Eq.(1), for Day1, Day2, Day3 and Day7 using design guides of AS3850.11. As per the calculated capacities for $N_{u,p}$ and $N_{u,c}$ shown in Table 3, concrete cone failure is the predicted failure mode and shows a lower strength than the pull-out strength for the tested anchors.

Table 3: $N_{u,p}/N_{u,c}$ for $h_{ef} = 50$mm, 60mm and 70mm at the tested hydration ages

<table>
<thead>
<tr>
<th>Day after cast</th>
<th>$f_{c,age}$ (MPa)</th>
<th>$h_{ef}=50$mm</th>
<th>$h_{ef}=60$mm</th>
<th>$h_{ef}=70$mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Day1</td>
<td>7.7</td>
<td>2.6</td>
<td>2</td>
<td>1.6</td>
</tr>
<tr>
<td>Day2</td>
<td>13.0</td>
<td>3.4</td>
<td>2.6</td>
<td>2.1</td>
</tr>
<tr>
<td>Day3</td>
<td>16.0</td>
<td>3.8</td>
<td>2.9</td>
<td>2.3</td>
</tr>
<tr>
<td>Day7</td>
<td>21.0</td>
<td>4.3</td>
<td>3.3</td>
<td>2.6</td>
</tr>
</tbody>
</table>

The physical testing showed that on Day1 and Day2, a pull-out failure mode was observed for each of the three anchor tests, of a similar shape as shown in Figure 4. These anchors were installed with void formers and had 50mm effective depth, $h_{ef}$, as shown in Figure 6 (a). The remaining anchors were exhibiting concrete cone failure mode, shown in Figure 3a. On Day3 and Day7, concrete cone failure, depicted in Figure 3a, was observed for all anchors at these time intervals. All ultimate failure loads were less than steel yield stress levels.

Figure 8 Typical Concrete Cone Failure to the surface of a headed anchor
In the experiment, a combined failure mode, concrete cone and pull-out, was observed for anchor with 60mm effective embedment for Day1, Day2 and Day3. In the experiment, concrete cone failure was observed for all anchors with 70mm effective depth, shown in Figure 8. Pull-out failure through the void, was observed for 50mm deep anchors at Day1 and Day2 after cast, Figure 10. ATENA, a nonlinear finite element (FE) program, was used to model the headed anchors with the void former. Two anchors were modelled with effective embedment depth of 50mm and 70mm. In both models, the concrete material with 13MPa compressive strength was used. Figure 9 and Figure 11 show the principal fracture strain and failure modes for FE models. FE analysis confirms that the pull-out failure mode through the void occurs due to the smaller energy required to create the fracture surface in a shape of pull-out cylinder versus a concrete cone, Figure 11.

Figure 9: Predicted Concrete cone failure for headed anchor with 70mm effective depth

Figure 10: A cylindrical fracture plane is tested at 50mm effective embedment depth
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Figure 11: Predicted Pull-out failure mode, showing the fracture plane through the void former for headed anchor with 50mm effective depth

The ultimate tensile load from each test, \( N_{u,test} \) was compared to the predicted failure load by Eq.(1), \( N_{u,c} \). Figure 12 and Figure 13 shows the ratio of \( N_{u,test}/N_{u,c} \) for different concrete compressive strengths for concrete cone failure mode and pull-out failure mode. Mean concrete compressive strength at time of test \( f_{c,age} \) was used instead of \( f'_{c,age} \) in Eq.(1) to calculate \( N_{u,c} \).

Table 4 gives the average, coefficient of variation (CoV) and the lower characteristic values for \( N_{u,test}/N_{u,c} \). The lower characteristic value is calculated using the normal distribution sample size factor, \( k_s \) denoted in AS3850.1. The sample size for the pull-out failure is six, Table 4 suggests the use of Eq.(1) for anchors with pull-out failure through the void can overestimate the anchor capacity. Further, Table 4 shows CCD method in AS3850.1 is suitable for use in predicting the failure load of headed anchors with shallow effective embedment depth that exhibit concrete cone failure mode.

![Figure 11: Predicted Pull-out failure mode, showing the fracture plane through the void former for headed anchor with 50mm effective depth](image)

![Figure 12: Ratio between observed ultimate load at test and ultimate predicted load by Equation (1) for concrete cone failure mode](image)
Conclusion and recommendations

Pull-out through the anchor void can occur in shallow embedded headed anchors, less than 70mm, and more predominantly in early age concrete, upto 3 days from the onset of hydration.

The CCD methodology4, 5 models show an acceptable accuracy in predicting the failure load of shallow anchors which exhibit concrete cone failure mode at lower than 28-day concrete compression strengths.

ATENA can be used to accurately predict pull-out failure mode of shallow embedded headed anchors in early age concrete and shallow anchor effective embedment.

In the use of a capacity model for shallow embedded headed anchors, it is recommended to introduce boundary conditions for the application of the CCD methodology, applied in AS3850.11 appendix B. The recommended CCD boundary conditions for anchor placement is 60mm $h_{ef}$, concrete compressive strength is 15MPa $f_{c,age}$ and anchor design is $(r_v / h_{ef}) \leq 0.5$. 

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Table 4: Average, CoV and characteristic values for $(N_{u,test}/N_{u,c})$

<table>
<thead>
<tr>
<th>Failure mode</th>
<th>Sample size</th>
<th>$K_s$</th>
<th>Average $(N_{u,test}/N_{u,c})$</th>
<th>CoV (%)</th>
<th>Characteristic $(N_{u,test}/N_{u,c})$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete cone failure</td>
<td>66</td>
<td>1.956</td>
<td>1.236</td>
<td>9.0</td>
<td>1.017</td>
</tr>
<tr>
<td>Pull-out failure through void former</td>
<td>6</td>
<td>3.091</td>
<td>1.062</td>
<td>4.4</td>
<td>0.917</td>
</tr>
</tbody>
</table>

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Figure 13: Ratio between observed ultimate load at test and ultimate predicted load by Equation (1) for pull-out failure mode
The applied stresses on the concrete would be reduced through an increase of diameter of the headed section of the anchor, which increases the effective embedment depth the anchor.

Further experimental and FE work on shallow embedded headed anchors is needed to establish a suitable design model for concrete pull-out.

## 5 Acknowledgement

Ramsetreid, ITW Australia, for their continued generous support of our experimental work

Cervenka Consulting, Prague, Czech Republic, for their assistance and extensive knowledge on non-linear numerical simulation of concrete structures

### References

1. AS3850.1, Australian Standard Prefabricated concrete elements - General requirements. AS 3850:2015. 2015: Standards Australia


INFLUENCE OF COMPRESSIVE LOAD REVERSALS ON THE FATIGUE LIFE OF HEADED STUDS IN CASE OF CONCRETE CONE FAILURE

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ABSTRACT

It is well known that the application of relatively small compressive load-reversals leads to the significant decrease of the tensile fatigue life of uniaxial loaded plain concrete (Cornelissen)1,2. This is explained by the mismatch of the crack faces during the load-reversals and by the interaction between the differently oriented cracks developed under tension and under compression. In the case of fasteners under alternating cyclic loads, the anchor plate or the component that is being fastened can lead to high pressure on the concrete surface. The resulting compression in the concrete below the anchor plate may interact with the developing tensile crack and for the above reasons this may lead to the significant reduction of the expected fatigue life of the fasteners if concrete cone is decisive. However, the alternating tension-compression loading case is currently not covered by the proposed fatigue verifications in prEN 1992-4 and subsequently, further studies are required to address this issue.

In this experimental study, the influence of compressive load-reversals on the fatigue life of concrete cone was investigated. To this end, preliminary fatigue tests were carried out on Ø25 mm single cast-in headed studs loaded in alternating tension-compression. To simulate the above described tension-compression realistically, a special loading fixture with a clearance hole was designed, which allowed the centric surface compression, whilst the tension loads were transferred into the headed stud via screwed connection. The tension-compression tests were carried out as constant amplitude tests with various upper loads (F_max/N_u,m) and constant compressive loads (F_min). The compressive load was targeted to reach 10% of the concrete compressive strength at h_e embedment depth, where the conic-shaped crack face initiates under tension. The results of the tension-compression tests were plot into Wöhler-diagrams and were compared to the results from tension-tension loading tests on headed studs (Lotze)3. The results are in a good agreement with those available for plain concrete. However, further experiments are required for a sufficient statistical evaluation.

1 Introduction

When fasteners are subjected to periodically or harmonically repeated loads, their fatigue resistance must be verified according to the provisions of EN1992-4. During the fatigue verification of fasteners, the fatigue behavior of all materials and anchor parts employed in the load transfer must be
taken into account. Furthermore, the fatigue resistance of all failure modes of fasteners under tension and under shear loading must be known to conclude for the fatigue life of a fastening system. Subsequently, the prEN1992-4 includes the fatigue verifications of the different failure modes under tension (steel failure, concrete cone, concrete splitting, concrete blow-out, pull-out) and shear loading (steel failure, concrete edge failure, pry-out). The provisions of EN 1992-4 cover the fatigue verification of post-installed and headed anchors under pulsating tension, under pulsating or alternating shear loading and the combined tension and shear loading is also taken into account by the corresponding conservative interaction formulas. However, the EN1992-4 has several limitations regarding the proposed fatigue verifications. Among others, the alternating tension-compression loading case is not covered by the recent proposals. Fasteners, however, are not only subjected to pulsating tensile loads, but also to alternating loads as well.

![Figure 1: Single fastener loaded in alternating tension and compression](image)

The tensile loads are transferred mostly into the fasteners via bolted connections (thread) and the loads from the fastener are transferred into the concrete via mechanical interlock, friction, bond or by the combination of these load-transfer mechanisms. The conically shaped tensile cracks initiate already at 30% of the ultimate concrete cone capacity. Fasteners, which are installed with the in-place or pre-positioned technique are designed with a clearance hole in the anchor base plate or in the element, which is being fastened. Consequently, during the compressive load-reversals, not the anchor itself, but the concrete is subjected to surface pressure, whereas the compression is transferred into to concrete over the fixture. This results in compressive stresses in the concrete. The effect of compressive minimum stresses on the fatigue life of the fasteners in the case of concrete cone failure was not investigated yet.

However, it is known that one of the most important parameters influencing the concrete fatigue life is the level of the minimum stress, particularly when the minimum stress is compressive and the maximum stress is tensile. Cornelissen observed the detrimental effect of compressive minimum stress on the tensile fatigue life of concrete. Uniaxial pulsating tension-tension and alternating tension-compression tests on Ø=120 mm, h=300 mm concrete cylinders showed that the maximum relative tensile stress reduces from 60% to 40% at 10⁶ loading cycles if only 5-10% of the concrete compressive strength as minimum stress is applied during the cyclic loading (Figure 2). This effect may even be more pronounced at higher number of cycles. This phenomenon by Cornelissen was explained by the mismatch of the tensile cracks under compressive load-reversals and the by the interaction of the developing cracks under tension and compression. The developing additional stresses on the crack faces during compressive load-reversals result in an accelerated material degradation and thus lead to the significant reduction of the fatigue life. Since fasteners generate
complex 3D stress concentrations in the concrete, the findings based on the fatigue test results under uniaxial loading conditions must be verified by new tension-compression tests on fasteners.

Figure 2: a) Test specimen for uniaxial tension-tension and tension-compression tests b) Test results (Cornelissen)\(^1\)

2 Scope

Since the concrete cone failure of fasteners is also considered as a concrete tensile failure, the detrimental effect of compressive load-reversals on the fatigue life of fasteners in case of concrete cone failure was expected. This assumption was made based on the comparison of uniaxial tension-tension and tension-compression tests (Figure 2). To confirm the effect of compressive minimum stress on the fatigue behavior of fasteners, tension-compression tests were carried on \(\Omega=25\) mm \(h_{cf}=50\) mm cast-in headed studs within the scope of this study. The tests were carried out as constant-amplitude tests with various maximum tensile loads and with constant compressive minimum load. The results of the executed fatigue tests were compared to the results of pulsating tension tests, which were carried out by Lotze\(^6\). Furthermore, the results of tension-compression and tension-tension tests were compared to the results of uniaxial fatigue tests from Cornelissen to emphasize the differences or similarities in the fatigue behavior of concrete under uniaxial conditions and under 3D conditions in case of fasteners.

3 Experimental investigations

3.1 Testing concept and test program

The compressive load reversals in the fastening technology can be caused by either dead or live loads and can be transferred into the concrete via the anchor base plate, provided that the anchor base plates are designed with a clearance hole. The headed stud is suitable for the investigation of the concrete cone in general because the location of the expected tensile crack initiation under tension is known and other failure modes (steel failure, pull-out) can be avoided. It was assumed that during the tensile phase of the cycling tensile crack develops (Figure 3a) which will be closed by surface compression (Figure 3b). Furthermore, it was assumed that the equally distributed surface compression spreads out under 45° in the unreinforced concrete and the compressive stress at \(h\) depth reads:
\[ \sigma_d = \frac{F}{(a + 2h)^2} \]  

where \( F \): total load applied on the load actuator, \( a \): side length of the quadratic anchor plate.

Figure 3: Determination of stresses in the concrete at the depth of tensile crack initiation (\( h_{ef} \))

It is well known that the crack propagates continuously during cyclic loading under 30-35° from the horizontal towards the concrete surface. The evidence for the assumed crack propagation is the continuous increment of the vertical anchor displacement observed during the fatigue tests, which shows a three-phase damage process. The three-phase damage process consists of (1) crack initiation, (2) continuous crack propagation, which is followed by an (3) unstable phase and fatigue failure. This was observed on concrete under uniaxial fatigue loading\(^1\)\(^2\) as well as on fasteners\(^6\). In the case of fasteners, the crack propagation means that the crack tip is shifting upwards under approximately 30°-35° during cyclic loading and therefore, the calculated compression at the crack tip is not constant; it increases with increasing number of cycles. Since the rate of the tensile crack propagation alongside the theoretical concrete cone under cyclic loading is not known, this effect was not taken into account during the evaluation and the applied compression was considered as a constant value during the cycling and was calculated according to Equation 1 at the effective embedment depth (\( h_{ef} \)). The application of 10\% of the compressive strength (\( f_c \)) at \( h_{ef} \) depth was targeted during the tension-compression tests. The test program with the targeted load levels is shown in Table 1.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Test type</th>
<th>targeted ( F_{max}/N_{u,m} ) [-]</th>
<th>( G (F_{min})/f_c ) at ( h_{ef} ) depth</th>
<th>Concrete age in days at testing</th>
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<tbody>
<tr>
<td>1</td>
<td>Static pull-out test</td>
<td>-</td>
<td>-</td>
<td>33</td>
</tr>
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<td>Static pull-out test</td>
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<td>-</td>
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</tbody>
</table>
The compressive strength as well as the concrete cone capacity, which the applied compression and maximum load are compared to, are not constant values. Therefore, static tests were carried out prior to the cyclic tests and after the execution of all cyclic tests. With increasing concrete age, the measured mean compressive strength of the concrete members increased from $f_{c,m,35\text{days}}=33.2\ \text{N/mm}^2$ to $f_{c,m,46\text{days}}=35.3\ \text{N/mm}^2$. The formula that calculates the compressive strength in function of the concrete age is given in Figure 4a. The required force ($F_{\text{min}}$) to ensure $\sigma=0.1\cdot f_c$ at the depth of the crack initiation ($h=h_{\text{ef}}$) was calculated based on Equation 1. The side length of the quadratic anchor base plate was $a=150\ \text{mm}$. The applied tension force was related to the static reference load (static concrete cone capacity), which was also measured repeatedly at a concrete age of 33, 39, and 49 days (Figure 4b). Although the mean static concrete cone capacity ($N_u$) increased from $N_{u,m,33\text{days}}=28.4\ \text{kN}$ to $N_{u,m,39\text{days}}=32.5\ \text{kN}$, it remained almost constant during the execution of the fatigue tests. Therefore, the mean value of the static pull-out tests at the age of 39 and 49 days was taken as reference $N_{u,m}=31.9\ \text{kN}$. The ultimate concrete cone capacity ($N_u$) and the concrete compressive strength ($f_c$) at different concrete ages are shown in Figure 4. Furthermore, the time-span of the executed fatigue tests is also indicated.

![Figure 4: Results of static tests at different concrete age: a) concrete compressive strength ($f_c$) and b) ultimate concrete cone capacity ($N_u$)](image)

### 3.2 Tested materials

For the tension-compression tests a special concrete specimen was developed. The concrete specimens of size 240x52x40 cm were cast in rigid formworks. With respect to the geometry of the testing machine and for economic reasons, 4 headed studs of $\Omega=25\ \text{mm}$ were cast in each concrete specimen. Two requirements must have met regarding the design of the concrete test member: (1) provide an unreinforced concrete area in the load-transfer area of the headed stud, (2) avoid excessive cracking. This can be caused by either the developing splitting forces or the bending moments according to Figure 5.
Supplementary reinforcing bars were assigned in the concrete members to take up the resulting high splitting forces and bending moments due to the high surface compression. The developing splitting forces were estimated according to the strut-and-tie model and the bending moments were determined conservatively (big span) assuming an uneven concrete surface on the bottom side of the specimen. During the assignment of the required longitudinal reinforcing bars and stirrups, the fatigue of the steel was also taken into account. The resulting reinforcement is shown in Figure 6. The concrete specimens were using one concrete batch and the concrete composition is shown in Table 2.

### Table 2: Concrete mixture

<table>
<thead>
<tr>
<th>Batch No.</th>
<th>Aggregates</th>
<th>Cement CEM I 32,5 R</th>
<th>Water</th>
<th>w/c</th>
<th>Slump test /consistency class</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>708 [kg/m³]</td>
<td>465 [kg/m³]</td>
<td>689 [kg/m³]</td>
<td>265 [kg/m³]</td>
<td>185</td>
</tr>
</tbody>
</table>

#### 3.3 Test arrangement

The fatigue tests were carried out using a Schenck servo-controlled hydraulic testing cylinder, which was built in a rigid steel frame (Figure 7a). The capacity of the actuator and the range of the built-in load cell was 400 kN for both compression and tension. The reaction loads of the applied tension loads on the fastener were transferred into the rigid testing table via the bracing of the concrete specimen. For the centric load application, a special rigid loading fixture was designed which allowed the equal stress distribution under compression as well as the centric tensile loads. The centricity of both the surface compression and the tension was ensured using 3 spherical calottes according to Figure 7. The tests were carried out at 4-6 Hz testing frequency and during the tests the load and the vertical displacement of the anchor was recorded continuously. The fatigue tests were launched according to the following procedure: As a first step, a minimum compression of 210 kN was applied on the specimen. This applied compression was kept constant in load-control until the nut below the middle spherical calotte was tightened. On one hand, this process allowed the...
alignment of the loading fixture to provide equal distribution of the compressive stresses below the loading fixture. On the other hand, the tightening of the nut lead to the prestressing of the two upper spherical calottes. This was required to eliminate the problems of the load-control when passing through zero load. The compression was reduced the middle load \((F_{\text{max}}+F_{\text{min}})/2\), which was a negative load (compression) in each case. After hand-tightening the nut on the headed stud, the sinusoidal cycling with the corresponding load amplitude was started at 1 Hz testing frequency, which was increased in 0.5 Hz steps up to approximately 4.5 Hz.

![Test setup](image)

**Figure 7:** Test setup a) servo-hydraulic testing cylinder in a rigid steel frame b) schematic of the loading fixture

### 3.4 Test results

All executed fatigue tests failed by concrete cone as it was expected and the typical failure mode is shown in Figure 8. The number of cycles at failure is contained in Table 3. The increment of the compressive strength was taken into account prior to each executed test in such a way, that the applied compression was increased accordingly to provide approximately \(0.1f_c\) at the embedment depth. This was necessary because the applied minimum load or stress must be defined to each Wöhler line in the Wöhler-graph. If the applied minimum load (or minimum stress) would deviate from each other in the case of each point contained in the Wöhler-graph, the points must not be connected with each other to obtain the Wöhler-curve. The adjustment of the applied compression at each test start lead to a deviation of 0.2% from the targeted compressive stress (Table 3).

![Concrete cone](image)

**Figure 8:** Typical failure mode – concrete cone
The first graph shows a comparison of the pulsating tension tests from uniaxial tests (Cornelissen)\textsuperscript{1,2} and the pulsating tension test results on headed studs (Lotze)\textsuperscript{6} (Figure 9a). This comparison can be also found in Ref. \textsuperscript{6} and it was found that the results measured on concrete cylinders are slightly below the results on fasteners in case of concrete cone failure. This confirmed the general and conservative applicability of the uniaxial tests results for fasteners. In Figure 9b, the new tension-compression results were compared to the uniaxial tension-compression results from Ref.\textsuperscript{1}. Note that in both cases the linear regression lines of Cornelissen according to Equation 2 (tension-tension) and Equation 3 (tension-compression) were used for comparison\textsuperscript{1,2}.

Table 3: Results of tension-compression tests (Headed studs, \(h_{ef}=50\) mm)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>(F_{\text{min}}) [kN]</th>
<th>(F_{\text{max}}/N_{u,m}) [-]</th>
<th>(F_{\text{min}}) [kN]</th>
<th>Targeted (G(F_{\text{min}})/f_c)</th>
<th>applied (G(F_{\text{min}})/f_c)</th>
<th>Concrete age in days at testing</th>
<th>N- number of cycles at failure</th>
<th>Failure mode</th>
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<tbody>
<tr>
<td>6</td>
<td>26.0</td>
<td>0.82</td>
<td>206</td>
<td>0.1</td>
<td>0.098</td>
<td>40</td>
<td>322</td>
<td>Concrete cone</td>
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<tr>
<td>7</td>
<td>24.4</td>
<td>0.76</td>
<td>206</td>
<td>0.1</td>
<td>0.098</td>
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<td>1212</td>
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<tr>
<td>8</td>
<td>22.8</td>
<td>0.71</td>
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<td>0.1</td>
<td>0.101</td>
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<tr>
<td>10</td>
<td>21.0</td>
<td>0.65</td>
<td>214</td>
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<td>10407</td>
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<tr>
<td>11</td>
<td>19.6</td>
<td>0.61</td>
<td>218</td>
<td>0.1</td>
<td>0.100</td>
<td>47</td>
<td>75333</td>
<td>Concrete cone</td>
</tr>
</tbody>
</table>

\[ \log N = 14.81 - 14.52 \frac{\sigma_{\text{max}}}{f_{cm}} + 2.79 \frac{\sigma_{\text{min}}}{f_{cm}} \]  \hfill (2)  

\[ \log N = 9.36 - 7.93 \frac{\sigma_{\text{max}}}{f_{cm}} - 2.59 \frac{\sigma_{\text{min}}}{f_{cm}} \]  \hfill (3)  

Figure 9: Comparison of the tension-tension results on concrete cylinder (Cornelissen\textsuperscript{1}) with tension-tension tests on headed studs (Lotze\textsuperscript{6}) and comparison of tension-compression results on concrete cylinder (Cornelissen\textsuperscript{1}) with the new tension-compression tests on fasteners.

As it was expected, the new results from the executed tension-compression tests in Figure 9b show the strong influence of the compressive minimum stress on the fatigue life of fasteners. In comparison to the pulsating tension results from Lotze, the relative load \((F_{\text{max}}/N_{u,m})\) reduces from 66\% to 46\% at \(2 \cdot 10^6\) number of cycles if only 10\% of the compressive strength is applied. The preliminary results show that the reduction in the fatigue life of fasteners in the case of concrete cone failure is less pronounced due to the presence of compressive minimum load than in the case of the
uniaxial tests. The proposed approach for the determination of the compressive stress at $h_{ef}$ depth allowed the qualitative and quantitative comparison of the new test results with the existing test results from uniaxial tests.

4 Conclusions and Outlook

The concrete fatigue behavior in tension is strongly influenced by the minimum stress applied, particularly when the minimum stress is compression. These findings are based on results from uniaxial tests, which were carried out on concrete cylinders by Cornelissen. In the fastening technology, fasteners are also subjected to alternating tension-compression, where the tension is generally transferred into the fasteners by means of bolted connection and the compression is transferred into the concrete over the anchor base plate or over the element which is being fastened. However, this loading case is not covered by the fatigue verifications of prEN1992-4 and the unfavorable effect of such compressive load-reversals on the fatigue life of concrete cone failure was investigated yet. Although, the number of the executed tests is not sufficient for statistical evaluation, the tension-compression tests show the significant decrement of the fatigue life due to the presence of relatively small compressive load reversals, which should be taken into account in the design. Further experiments are required for the better statistical evaluation of this problem, which should lead to the extension of the proposed fatigue verifications in EN1992-4.

References:


2. Cornelissen, H.A.W.: Fatigue of plain concrete in uniaxial tension and in alternating tension-compression, Delft University of Technology, Department of Civil Engineering, Report No. 5-81-7, 1981.


BONDED ANCHORS
BACKGROUND, REGULATIONS AND LATEST DEVELOPMENTS OF POST-INSTALLED ADHESIVE ANCHORS AND REBARS

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ABSTRACT

Post-installed adhesive anchors and rebars are getting increasingly popular to connect structural and nonstructural components to concrete or to connect new and existing reinforced concrete structures. To this end, threaded rods or reinforcing bars are bonded to the hardened concrete by means of adhesives which are generally two-component chemicals filled in injectable cartridges. Over the years, the test program required for a full product qualification has been extensively extended and the design concept has been conclusively completed to ensure a safe connection in all possible load configurations and installation situations. The underlying qualification guidelines and design codes are largely harmonized in Europe and the USA. However, there are also differences in some details e.g. in regard to seismic design and fire design. Moreover, the regulative landscape for the qualification and design of post-installed anchors and rebars experienced some formal changes, causing easily confusion.

This paper introduces the context of post-installed adhesive anchoring systems for post-installed anchors and rebars. An overview of relevant international regulations for design and qualification is provided, and specific requirements of European and US regulations are highlighted. The differences between post-installed anchor and post-installed rebar applications are briefly outlined and some of the latest research and developments presented. The contribution will help to bring adhesive anchoring systems to the attention of a broader audience of the engineering and research community, and to increase a better understanding of the technical background.

1 Introduction

Post-installed adhesive anchoring systems consist of a steel element and an adhesive to bond the steel element to the concrete base material. The steel element is in general a threaded rod or a reinforcing bar. Adhesive anchors are used to connect structural and nonstructural elements to the existing concrete structure. Post-installed rebars also allow upgrading and retrofitting of reinforced concrete structures by connecting new concrete to existing concrete. In many design cases, the concrete cannot be assumed to remain uncracked in service because the very small tensile capacity of concrete results in immediate cracking when the concrete element is loaded in tension. Since the anchor performance is substantially affected when located in a crack, adhesive anchor products can either be qualified for uncracked concrete only or for uncracked and cracked concrete.
The adhesives used for concrete anchors are generally two component adhesives packaged in either cartridges or capsules which separate the components prior to installation. Today, most of the adhesives are sold in cartridges allowing the injection of the exact adhesive quantity required through a mixing nozzle directly into the hole drilled for installation. The cartridges are either side-by-side or coaxial type, and their specific sizes reflect the mix ratio of the chemical formulation. Most of the commercially available adhesives can be broadly categorized as fast-setting systems, based on vinylester resin or similar, or slow-setting systems, based on epoxy resin, which hardening processes are polyaddition or polymerisation, respectively. Formulations with cement content are often called hybrid adhesives, combining the advantages of chemical and mineral binders. Example components of adhesive anchoring system are shown in Fig. 1.

![Figure 1: Examples of adhesive anchoring systems (not to scale): a) Capsule (vinylester formulation in glass), cartridge (side-by-side, epoxy formulation, mixing ratio 3:1; co-axial, hybrid formulation, mixing ratio 10:1); b) Mixing nozzle and dispensing tool for use with cartridges; c) Threaded rod with washer and nut, reinforcing bar](image)

Advantages of adhesive anchors compared to mechanical anchors include: high flexibility of diameter and embedment depth and thus load capacity, compatibility with different steel elements and grades, small anchor spacing and edge distances due to small expansion forces, and sealing against water ingress. Conversely, adhesive anchors generally take longer to install, can be more sensitive to service temperature and potential creep failure over time which must be thoroughly addressed in the qualification and design process.

During past fifteen years, substantial research and development have been made to increase the fields of applications and the safety of post-installed adhesive anchors. Frequently updated qualification and design guidelines reflect the latest state-of-the-art. Third party approval documents from agencies such as ICC-ES, CSTB or IEA provide design data for the conditions the anchor has been qualified for. Approved products guarantee a high level of safety due to the regulated system of qualification, approval and design. Worldwide, designers and specifiers show an increased awareness for approved products and their benefits in view of safety and performance.

Adhesives may be qualified for post-installed adhesive anchor applications or post-installed rebar applications or both. In case of post-installed rebar (PIR) applications, reinforcing bars are used for the connection of new concrete elements to the existing concrete structure which are designed
Philipp Mahrenholtz and Jake Olsen

according to the structural concrete design codes for cast-in-place rebar. In contrast, adhesive anchors are designed according to anchor design rules similar to mechanical anchors or headed bolts. Accordingly, the qualification program of adhesives for PIR applications differ significantly to that of adhesives for anchor applications. Today, the European and US regulations for qualification and design of anchors and PIRs are to a large degree harmonized. An overview of the relevant standards is provided in Table 1. In the following sections, specific aspects for the approval, the design and the qualification are discussed.

Table 1: Key regulations for qualification and design of adhesive anchors in Europe and the USA

<table>
<thead>
<tr>
<th></th>
<th>Europe</th>
<th>USA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Anchor application</td>
<td>PIR application</td>
</tr>
<tr>
<td>Technical approval</td>
<td>ETA</td>
<td>ESR (ICC-ES), ER (IAPMO)</td>
</tr>
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<td>Qualification standard</td>
<td>previous ETAG 001 Part 5</td>
<td>EOTA TR 023</td>
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<tr>
<td></td>
<td>current EAD 33-0499</td>
<td>EAD 33-0087</td>
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<tr>
<td>Design code</td>
<td>previous EOTA TR 029</td>
<td>EN 1992-1-1</td>
</tr>
<tr>
<td></td>
<td>current EN 1992-4</td>
<td></td>
</tr>
</tbody>
</table>

2 Technical Approval

Construction products may get technically approved by an independent third party which review the test and evaluation reports, and issue the approval documents. This product qualification enables the planner to carry out a safe and professional design for the anchorage. The required design data and further specifications are laid out in the approval documents. This system ensures that the anchors, as integral and permanently remaining parts of the structure, comply with the legal requirements for construction products and are able to perform as certified during its lifetime which is in general assumed to be 50 years as for structures. This high standard results in international recognition beyond the boundaries of Europe and the USA. For clarity it is noted that technical approvals are referenced by other terms, e.g. “product specification” in some European standards or “evaluation reports” or “listings” by some US agencies. However, in this contribution the term “approval” is generally used for reasons of simplicity.

In Europe, national product approvals have been available already for several decades and remain meaningful for regional product applications. However, European Technical Approvals (ETAs) have been increasingly established since the turn of the millennium. ETAs are meant for construction products not covered by a harmonized standard, as this is the case for anchors. They are issued by any of the designated Technical Assessment Bodies (TABs) which are in the first place the national building authorities. Europe has seen some regulatory changes over past years due to the transition of the Construction Product Directive (CPD) to the Construction Product Regulation (CPR) in 2013. In this course, the European Technical Approvals turned into European Technical Assessments (also abbreviated ETAs) which however are technically the same. In conjunction with an independent
factory control, ETAs allow the manufacturer to place a CE marking on the product. This system for the Assessment and Verification of Constancy of Performance (AVCP) safeguards the reliability of the Declaration of Performance (DoP) which every manufacturer has to publish for all products holding an ETA (Fig. 2a).

In the USA, the most renowned approvals in the field of concrete anchors are the Evaluation Service Reports (ESRs) from the Evaluation Service of the International Code Council (ICC-ES) and to a lesser extent, the Evaluation Reports (ERs) from the International Association of Plumbing and Mechanical Officials (IAPMO). PIRs are treated the same as any other anchor product. Where applicable, performance data for PIR applications are listed in the same ESR or ER as those for adhesive anchor applications. Due to a separate qualification guideline, the PIR approval is in Europe generally covered by a separate ETA.

Adhesives from companies doing business internationally are often furnished with both, European approvals (ETAs) and US approvals (ESRs or ERs). In addition, some adhesives are listed on Qualified Product Lists (QPLs) of the US Departments of Transportation (DOT) of the individual US states. These listings, however, typically require limited testing of the adhesive at a lower technical level and there are attempts to consolidate their requirements by replacing them gradually by those stipulated in ACI 355.43. Next to these anchor performance approvals, adhesive anchors can have a number of further certifications. The adhesive can be certified in regard to harmlessness to water, e.g. by the National Sanitary Foundation (NSF) or Water Quality Association (WQA). Further, the adhesive can be tested in regard to volatility of their compounds, e.g. for a VOC or LEED certificate. Moreover, the chemical substances of adhesives are potentially harmful and therefore have to comply with the Global Hazard Safety (GHS) regulations (e.g. 1272/2008 EWG4 or OSHA 29 CFR 1910.12005). Representing all these approvals and certificates on cartridge labels of premium products, together with further installation instructions, results in some congestion on the label (Fig. 2b). Material Safety Data Sheets (SDSs) provide detailed information on the chemical compounds.

Figure 2: a) Example of a Declaration of Performance (DoP); b) Example of a cartridge label showing relevant icons to represent approvals and listings, as well as installation instructions.
3 Qualification

In Europe, ETAs are issued on the basis of European Assessment Documents (EADs). The EADs are developed by the European Organization for Technical Assessment (EOTA). Before the introduction of the CPR in 2013, ETAs were issued on the basis of European Technical Assessment Guidelines (ETAGs) defining test conditions and assessment criteria. These ETAGs continue to be used till the respective EADs are formally introduced. For adhesive anchors, the relevant ETAG was the ETAG 001 Part 5\(^6\) which was also taken as the starting point for the EAD 33-0499\(^7\) succeeding in 2016. For PIRs, the qualification document used to be the EOTA Technical Report TR 023\(^8\), what becomes now the EAD 33-0087\(^9\).

In the USA, ASTM E488\(^10\) defines the test standards for anchor qualification. Where test conditions are not defined and since updating of ASTM standards takes very long, the American Concrete Institute guideline ACI 355.4 fills the gap and also provides assessment criteria for adhesive anchors. The ACI 355.4 is supplemented by the ICC-ES Assessment Criteria document AC308\(^11\), which may be revised several times a year if new research requires this. Starting with the 2013 revision, qualification of adhesives for PIR applications is possible according to AC308 Table 3.8.

For adhesive anchors, creep tests are of particular importance. Creep is a phenomenon observed for polymers which deform under sustained loads. This behavior is tested by sustained load tests (Fig. 3a) which often determine the maximum bond stress the product is approved for. Product evaluation is based on the creep displacement extrapolated for an assumed service life of 50 years. Creep rates depend on the ambient temperature: The higher the temperature the larger the creep rate. For this reason, adhesives are tested and qualified for various service temperature ranges. Also the curing of the adhesive is temperature and product dependent: The lower the temperature the more the chemical reaction is inhibited, resulting in longer curing times and/or lower strengths. The goal of product development and qualification is to have a temperature behavior as stable as possible for elevated temperatures (Fig. 3b).

![Figure 3: a) Test setup for a creep test with an adhesive anchor (Mahrenholtz et al.\(^12\)); b) Comparison of the bond strength of various adhesive products at standard and elevated temperature (Hoermann-Gast and Olsen\(^13\))](image-url)
4 Design

In Europe, the design of bonded anchors has been carried out according to the EOTA Technical Report TR 029 for more than a decade. An advanced development of the design rules stipulated therein was introduced as pre-standard CEN/TS 1992-4 in 2009\textsuperscript{14}. Based on this pre-standard, the process of creating an anchor design standard within the Eurocode (EC) series for structural concrete design (EC2) was finally completed by the imminent implementation of the Part 4 of the EC2, i.e. EN1992-4\textsuperscript{15} in 2017. Design of PIRs and their development length was always conducted according to the general structural concrete design code EN1992-1-1\textsuperscript{16}.

In the USA, the International Building Code (IBC\textsuperscript{17}) references adhesive anchors since the 2011 edition of the ACI 318\textsuperscript{18} standard which is the design basis for the IBC. Anchor design rules were given in the Appendix D of the ACI 318. With the 2014 edition, this appendix turned into the Chapter 17. The development lengths of PIRs are calculated according the rules laid out in Chapter 25. The American Association of State Highway Transportation Officials (AASHTO), a standard setting body for civil structures which voting members are the DOTs of all US states, used to have their own design standard (AASHTO LFRD\textsuperscript{19}), but is currently heading towards incorporating the ACI 318 design provisions for adhesive anchors.

With the IBC 2012 code change, room (i.e. standard) temperature is not a possible design temperature anymore as its adherence during the entire life can hardly be guaranteed in practice. Minimum design temperature in the USA is now 110°F, i.e. 43°C (elevated temperature). Moreover, the 2014 edition of the ACI 318\textsuperscript{20} and the EN 1992-4 substantially reduces the design strength for adhesive anchors under sustained tension load by a sustained load factor of 0.55. Because anchor design is still subject to substantial modifications, is very complex and requires product specific parameters, the use of special anchor design software is widely spread. It supports the designer and receives the necessary updates (Fig. 4a). The other software trend is Building Information Modelling (BIM) which is increasingly expanding into the anchor design application. Here, the anchor is not considered as a stand-alone item but as part of the complete building. Smart design tools allow for example the combined like conflict clash detection and integrated property markers (Fig. 4b).

Figure 4: a) Web based anchor design software on a mobile device (PDA\textsuperscript{21}; b) Revit plugin for designing anchors, support and bracing (HangerWorks\textsuperscript{22})
5 Accessories

Adhesive anchors made of a piece of commercially available threaded rod and some adhesive may appear simple on the first sight. However, a proper installation requires several additional accessories (Fig. 5) which are sometimes neglected in practice. Selecting the right accessories is very important as inappropriate accessories may cause trouble during drilling, cleaning, and adhesive injection, in particular for PIR applications in deep holes. Today’s premium adhesives permit design with very high bond strengths which can be achieved only when installations are conducted strictly according to the published instructions with the correct steps and equipment.

![Figure 5: Typical accessories used for adhesive anchoring systems (not to scale): a) Battery dispenser; b) Carbide drill bit; c) Diamond coring bit; d) Hollow drill bit; e) M-class vacuum; f) Hand pump; g) Wire brush with SDS connector; h) Injection tube extension; i) Piston plug](image)

Adhesive anchors are not as sensitive to drill bit tolerances or irregular boreholes as mechanical anchors. Still, adhesive anchors are qualified for specifically assumed adhesive thickness transferring the load from the steel element to the concrete base material by bond. The thickness of the adhesive is determined by the annular gap between the threaded rod or reinforcing bar and the borehole walls. For a given steel element, the gap width is defined by the used drill bit which diameter might comply with the dimensional requirements of relevant standards like the ANSI B21223 or the ISO 546824.

Adequate cleaning of the drilled hole is of paramount importance for adhesive anchors to achieve the designated strength. If the borehole wall is not properly cleaned, the adhesive cannot stick properly to the concrete base material. Installation training launched in the last decade increased the awareness, particularly the ACI’s adhesive anchor installer certification program which is now mandatory in the USA for certain critical applications. The cleaning procedure is specifically defined in the Manufacturer’s Published Installation Instructions (MPII) as a sequence of cleaning steps, generally consisting of a number of blowing and brushing actions. Careful cleaning is a time consuming procedure and requires, besides a hand pump or compressed air equipment, brushes...
which gradually wear out below the required diameter. To accelerate installation, hollow drill bits attached to a vacuum, which suck the drill mill during drilling, become increasingly popular. If qualified, hollow drill bits can eliminate the need for additional cleaning. This method does not only save time but is also less sensitive to misuses. Moreover, hollow drill bits help to reduce the dust exposition of the installer and are therefore recommended by the Occupational Safety and Health Association (OSHA\textsuperscript{25}) in the USA. In Europe, health and safety regulations are made at a national level. E.g. in Germany, the allowable maximum concentration of dust at the workplace is limited to a level which requires the use of so-called M-class vacuums (TRGS 504\textsuperscript{26}).

In case of deep embedment depths, extension hoses or tubes are mounted to the static mixer which allow injection of the adhesive starting from the bottom of the borehole. Using piston plugs ensures that no air is entrapped and the hardened adhesive is free of any voids. With increasing extension length, the required dispensing pressure increases. Increased flow times might also conflict with the gel time. For these reasons, powered dispensers, either battery or pneumatic type, are generally required for PIR applications with their typically deep embedment depths and large adhesive volume. Since the dispensing pressure as well as the setting time is product and temperature dependent, the right choice of a suitable adhesive product is critical. Diligent planning and familiarization with the equipment ensures a successful installation.

### 6 Seismic and Fire

In regard to seismic and fire qualification and design provisions for adhesive anchoring systems, the situation differs for Europe and the USA. While in Europe attention has been historically paid to fire but to seismic aspects since a relatively short time only, in the USA seismic regulations have been established for quite a while but fire is still not regulated. An overview is given in Table 2.

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For the past decade, research on seismic applications of anchors was one of the key research focus. Europe made a leap-frog in 2013 for seismic qualification and design of anchors with the implementation of the Annex E of the ETAG 001\textsuperscript{27} and EOTA TR 045\textsuperscript{28}, respectively, introducing...
two seismic performance categories, C1 and C2. These documents are about to be replaced by EOTA TR 049\textsuperscript{29} and the Annex C of the EN 1992-4\textsuperscript{30}, respectively. C1 qualification corresponds to the seismic qualification according to ACI 355.4, C2 qualification is based on advanced test load protocols with increased test crack widths. The requirement for C1 versus C2 design is still subject to an ongoing debate and will be regularized in the national annexes of the EN 1992-4. Seismic design of PIRs is carried out according to EN 1992 in conjunction with EN 1998\textsuperscript{31}. Qualification of PIRs for seismic applications is still waiting for getting finally regulated, which is assumed to be the case in near future. In the USA, seismic qualification of anchors and PIRs is covered by ACI 355.4, and their seismic design by ACI 318.

The development of ETAG 001 Annex E involved extensive numerical and experimental investigations. Anchor performance under realistic seismic conditions was studied by means of simulated seismic tests and benchmarked against dynamic shake table tests (Mahrenholtz\textsuperscript{32}). Seismic testing for C2 qualification requires complex servo-hydraulic test setups which allow the simultaneous control of anchor load and crack width (Fig. 6). Extreme crack widths of 0.8 mm, compression of the concrete member during cyclic testing, and stepwise increasing loading protocols are some of the used features.

The research focus is increasingly expanded to anchors and PIRs under fire attack. Fire design of anchors is straightforward and based on the rules defined in EN 1992-4 for cold design, and its informative Annex D giving additional advice for fire design. The design strengths corresponding to the concrete, steel, and pullout (bond) failure modes are generally reduced due to material deterioration during fire exposition. The fire resistance of anchors is in general tested by fire chamber tests according to the EOTA TR 048\textsuperscript{33}. In the past, regulated fire rating for PIRs was only possible for a German national approval which fell back on test conditions as laid out in the earlier EOTA TR 020\textsuperscript{34}. This test setup was taken over by the EAD 33-0087\textsuperscript{9} and fire rating is now provided within the ETA. Fire design of PIRs is conducted according to EN 1992-1-1 in conjunction with EN 1992-1-2\textsuperscript{35}. In the USA, there are no regulations for fire design and qualification available.
For a better understanding of the behavior of PIR connections under fire, large scale tests are essential as they help to validate the theoretical models of PIRs behavior under realistic fire conditions. One outstanding test program in this context is the Vulcain Project currently conducted at the French national building agency CSTB. In the presented example, a cantilever slab (2940 x 2000 x 150 mm) is connected to the wall by PIRs (8 pcs, 16 mm diameter, 135 mm embedment depth) using the Dewalt Pure150 adhesive (Fig. 7). The test specimen successfully endured an ISO 834-1 fire and failed at the predicted time of 120 minutes.

Figure 7: Fire testing: a) Test setup used for the Vulcain testing; b) Close-up of the concrete slab before testing (courtesy of CSTB)

7 Summary and Conclusions

Adhesive anchoring systems are a premium choice for post-installed anchorages of structural and nonstructural elements to concrete. Further, the adhesive anchoring systems can be used for post-installed rebar applications where new concrete elements are connected to existing concrete structures. Every adhesive holding an ETA or ICC-ES approval underwent an extensive qualification test program to ensure that it is suitable for the intended use and to allow a professional design. The complete system of qualification, approval and design ensures a high level of safety during the entire service life of 50 years minimum. Also other regions outside of Europe and the USA rely on this system and recognize or even require either ETA or ICC-ES approvals in their building code requirements, e.g. Australia and New Zealand.

Installing the anchor or rebar according to the installation instruction is important to ensure that the anchor or rebar develops the strength the anchoring was designed for. Using the right accessories is crucial for an efficient and safe installation. Particular attention has to be paid to an adequate cleaning where hollow drill bits help to reduce the effort substantially. Recent key research areas have been seismic applications and connections subject to fire exposure. More research is required for multi hazard assessments where the anchoring is subjected to both, seismic and fire loading.

The views expressed in this paper are the views of the authors only and do not necessarily reflect the views of Dewalt / Stanley Black & Decker, Inc.
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EXPERIMENTAL STUDY ON THE BEHAVIOR OF BONDED ANCHORS IN THERMALLY DAMAGED CONCRETE

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ABSTRACT

In our paper we analyzed the load bearing capacity of bonded anchors placed in thermally-damaged reinforced concrete. Our primary goal was to facilitate the reinforcing techniques of reinforced concrete structural elements damaged in fire events. For the tests, 8 mm diameter threaded rods installed with epoxy adhesive were used, with an embedment depth of 50 mm. Three levels of thermal loading (200, 300, 400 °C) were applied to the concrete specimens, with respect to the embedment depth. The mean value of the compressive strength of the concrete used in the tests was $f_c = 45.29$ N/mm².

1 Introduction

In case of thermally damaged buildings, fastening elements should be occasionally inserted in the reinforced concrete structures damaged by the fire. This may be required due to the replacement of the existing fasteners that have been damaged in the fire or to ensure a combined function of the additional elements together with the existing thermally damaged reinforced concrete structural elements (e.g. column jacketing, strengthening of the tension boom, and reinforcement of the slab with sprayed concrete). There are very few experiments dealing with the determination of the load-bearing capacity of fasteners placed in thermally-damaged reinforced concrete⁴, which became our topic of interest.

1.1 Fire resistance of concrete

The mechanical properties of concrete are changed by the effects of fire. Related research has been carried out since the 1940s⁴ and it is still actual. Previous studies indicated that changes in the mechanical properties of concrete are caused by a variety of chemical and physical changes that occur at different temperatures⁶. The extent of such changes is highly affected by the composition of the concrete. Changes in the material structure depend on the following factors⁷:
The change in volume of the various components of concrete varies as a function of temperature. The cement stone and the aggregate react to temperature increase in completely different ways. Consequently, the primary reason for the change in the strength of concrete is the deterioration of the connection between the aggregate and the cement stone. This effect is amplified by the dehydration of calcium dioxide and ettringite deposited on the surface of the aggregate particles. During thermal loading, as the temperature increases, the compressive strength of concrete is reduced together with the load capacity. Figure 1 clearly shows that the diagrams of concrete strength and deformation are getting flattened with increasing temperature. This means that lower strength values are linked to a larger deformation.

![Stress-strain relationship for concrete with quartz gravel aggregate as a function of temperature](image)

**Figure 1:** Stress-strain relationship for concrete with quartz gravel aggregate as a function of temperature

In *EN 1992-1-2:2004*, reduction of concrete strength in case of high temperatures is characterized by a reduction factor. The evaluation of this factor as a function of temperature is represented in Figure 2.
1.2 Fastening systems

Several post-installed anchors are available with different methods of load-transfer. The commercially available fastenings can transfer the load to the host material via the following mechanisms: mechanical interlock, friction or bond (Figure 3). Furthermore, the most recent techniques use combined bond and friction (e.g. bonded expansion anchors). In the case of expansion anchors, the load is transferred by friction. Generally, an expansion sleeve is expanded by an exact displacement or torque applied on the anchor head during the installation process. Chemical fastenings are anchored by bond. Bonded anchors can be divided into two subgroups: capsule or injection systems. The bond material can be either organic, inorganic or a mixture of them. In this case the loads are transferred from the steel (normally a threaded rod, rebar) into the bonding material and are anchored by bond between the bonding material and the sides of the drilled holes\textsuperscript{12,13,14}.

Load bearing of fastenings can be determined by taking the minimum of ultimate loads corresponding to different failure modes. In case of tensioned anchors steel failure, concrete cone failure, pull-out failure and splitting can occur (Figure 4).
Steel failure depends on the tensile strength of the steel rod. Steel capacity can be calculated from the ultimate steel strength ($f_u$) and the cross-sectional area ($A_d$)\textsuperscript{12,13,14}.

The properties of concrete cone failure mostly depend on the effective embedment depth ($h_{ef}$) and concrete strength ($f_{ck}$). Cone failure is the optimal failure type, because concrete strength is completely utilized. It can be either a full cone type or a partial cone type. The ultimate tensile force corresponding to full cone failure can be calculated as:

$$N_{u,k} = k * \sqrt{f_{ck}} * h_{ef}^{1.5}$$  \hspace{1cm} (1)

where coefficient $k$ shows whether concrete is uncracked ($k = 10.1$) or cracked ($k = 7.2$).

Partial cone failure is a common failure type of bonded anchors; in this case the bond between the bond material and concrete is partly damaged. This means a transitional failure type between cone failure and pull-out\textsuperscript{12,13,14}.

Pull-out failure has to be discussed separately for bonded and expansion anchors. Pull-out failure of mortar bonded anchors means bond failure between mortar and concrete, while pull-out failure excluding mortar means bond failure between the steel fastening and the bonding material. The bond strength ($\tau_{u,k}$) depends on the certain product, but its value is included in the corresponding approvals. Resistance against pull-out type failure can be determined as:

$$N_{u,k} = \pi * d * h * \tau_{u,k}$$  \hspace{1cm} (2)

where $d$ is the diameter of the anchor, $h$ is the embedment depth and $\tau_{u,k}$ is the bond strength of the bond material\textsuperscript{12,13,14}. Pull-out failure in case of expansion anchors is possible under tension, including or excluding the expansion sleeve (pull-out/pull-through).

Splitting failure is caused by reaching the critical edge-, spacing distances. Load bearing capacity can be influenced by distances from edges and by spacing distances; these effects can be taken into account by reduction factors\textsuperscript{12,13,14}.
2 Experimental program

In our experiments, we analyzed the load bearing capacity of anchors placed in thermally damaged reinforced concrete as a function of thermal load. In the tests carried out earlier\textsuperscript{1,2,3,4}, undercut fasteners were used. These studies did not address the behaviour of other types of fasteners, and that is why we used bonded anchors in our investigations to test the load bearing capacity of the bonded connection and its damage. During the experiments, the specimens were exposed to fire load on one side until they reached the desired temperature, then they were allowed to cool down at laboratory temperature (20 °C). The day after the fire load, typically after 24 hours, when the specimen had been cooled down, the fastener was inserted in the thermally damaged specimen. In order to allow the cross-linking of the adhesive, loading of the fasteners took place after a further 24 hours.

2.1 Tested anchors

One type of bonded anchor systems (epoxy resin) was tested. Bonded anchors were installed according to the MPII (Manufacturer’s Printed Installation Instructions). The embedment depth was $h_{ef}=50$ mm (≈6d, where “d” is the diameter of the anchor), and the diameter of the threaded rods was 8 mm, the strength class of threaded rods was 10.9.

2.2 Concrete mixture

The concrete mixtures were made by Portland cement (CEM I 42.5 N). The aggregates were natural quartz sand and quartz gravel and a superplasticiser of BASF Glenium C323 Mix was also used. The specimens were held under water for 7 days and then kept at laboratory temperature (20 °C) for an additional 21 days.

Compressive strength properties were tested on additional 3 cubes of 150x150x150 mm. Uniaxial compressive strength tests were carried out on concrete cubes 28 days after casting. The results were evaluated in accordance with EN 12390 -3:2009\textsuperscript{16} for concrete. The mean value of the mixture was $f_c = 44.79$ N/mm\textsuperscript{2}.

The dimensions of concrete specimens for pull-out tests were 300x300x150 mm.

2.3 Thermal loading

In laboratory tests, the concrete specimens were exposed to thermal load on one side. An electric furnace was used to produce this thermal load, with a heating curve shown on Figure 5. Based on the measurement data, the curve of the furnace is different from the standard fire curve according to ISO 834-1\textsuperscript{17}, so the experiments cannot be called a standard test. However, the heating curve of the furnace remained unchanged even after several checks, so it was well suited for the comparison of the specimens with varying degrees of thermal load, as well as for the preparation of a possible future standard test.

Thermal loading of the specimens was carried out in three different thermal steps relative to the embedment depth: 200 °C (Test 1), 300 °C (Test 2), 400 °C (Test 3) (Figure 5). Three specimens were tested after being subjected to each of the thermal steps. Temperature was measured in the face of specimen (T2) and in the embedment depth (T1) by a thermocouple. In the case of the embedment depth the thermocouple had been placed from the “cold” side of the specimen through an inspection...
hole (Ø 6 mm). Figure 5 shows that the specimen is gradually warmed, with a significantly slower tendency than the furnace. After reaching 100 °C the temperature increases for a short time, then water in the concrete vaporizes and starts flowing out from the concrete. The temperature does not rise because the heat energy is entirely devoted to the change of state of the water. The arrangement for the thermal loading test is shown on Figure 6.

![Figure 5: Temperature increase](image)

![Figure 6: Arrangement of heating for the specimen](image)

We observed no spalling of the concrete in any of the specimens during the tests. This may be due to the fact that water vapor generated in the concrete was not accumulated in the specimen due to the dimension of the specimen and the arrangement of the measurement, thus no moisture barrier was created. The steam was able to flow out freely though the test hole made for temperature measurement and the sides of the specimen. Therefore, the results of the test can be used only in cases where spalling does not occur in the reinforced concrete structure during the fire.

### 2.4 Pull-out tests

Our unconfined test setup is shown in Figure 7. The loading device was displacement controlled test apparatus, which allowed the recording of residual strength. This setup enabled the formation of all possible failure modes, the results were not affected by the geometry of the investigated samples (thickness of the test member, critical edge, placing). The measurement setup was capable to measure, record and show the applied load and related displacement of the anchor in real-time. The perpendicular pin-joints ensured the centrality of the acting force. The displacement was measured by two electronic transducers. Three additional independent displacement transducers were used to record the deformation of the surface. The load was measured by a calibrated load cell. The tests were carried out in accordance with the instructions given in *ETAG 001 Annex A*. The support distance was greater than 4 hₜₑᵢفاء.
3 Thermal analysis

Finite element analysis was used for the calculation change of temperature in the concrete specimens. The analyses were made in ANSYS Workbench 16.2 software. We used material properties (temperature dependent density, heat capacity, thermal conductivities and enthalpy) taken from the current Eurocode standards. Full-size finite element models of one quarter of the specimens were created by using axis-symmetric boundary conditions. Temperature values were measured at 11 points (on the surface of the specimen, at the embedment depth, and in 9 points between the surface and the point at the embedment depth).

4 Results

During the experiment, anchors have failed in all cases with a concrete cone failure. These failures illustrate that an adhesive bond can be created between the adhesive and the thermally stressed concrete with a strength that caused a concrete cone failure.

During the tests, no specimen showed either a clear pull-out failure or the combination of concrete cone failure and pull-out failure. On the surface of the concrete cones, aggregate particles close to the thermally stressed surface had a reddish discoloration, and the ratio of discolored particles increased when approaching the embedment depth and with increasing temperature. This discoloration can be explained by the chemical processes occurring in the quartz gravel. In case of thermally stressed specimens, the crack creating the concrete cone was just running in the cement stone, while aggregate particles remained intact. The aggregate particles could be easily twisted from their positions as a consequence of damage to the adhesion between cement stone and aggregate.

Figure 8 shows the tensile resistance of the anchors, while Figure 9 indicates the relative residual resistance values in function of the temperature. In Figure 9 each point of the curve means the average of three measurements, that are connected by spline fit.
Based on the results of the numerical analyses, the isotherm lines can be determined. Changes in temperature in the layers between the surface and the embedment depth (50 mm) are represented in Figure 10. Based on the reduction factors taken from EN 1992-1-2:2004 (see Figure 2), reduced compressive strength distribution can also be determined from the temperature distribution (Figure 11).

Load bearing capacity of anchors can be estimated by application of the isotherms and the calculated strength distribution. Weighted averages of the strength values represented in Figure 11 are the following: 33.09 N/mm² in case of Test 1, 25.12 N/mm² in case of Test 2 and 19.97 N/mm² in case of Test 3. These calculated strength values can be applied in Eq. (1) to determine the load bearing capacity of the anchors. During fire, due to the temperature load, cracks occur in the concrete, therefore coefficient $k$ in Eq. (1) is 7.2 that is the value that corresponds to cracked concrete.
Calculated and measured values of the load bearing capacity of bonded anchors are summarized in Figure 12.

If measured and calculated load bearing capacities are compared, visible that the calculated values overestimate the measured resistance. Measured resistance values of bonded anchors installed in thermally-damaged concrete are smaller than capacity values calculated by the average strength values corresponding to isotherm lines. Ratio of the calculated and measured resistances are around 1.6.

The force-displacement curves of pull-out tests performed on the standard specimens describe a brittle failure (Figure 13). The initial rapid force uptake, after reaching the maximum load, was followed by a rapid failure with small displacements. The force-displacement curves of pull-out tests performed on the thermally stressed specimens show a gradual decrease in load bearing capacity. It can be observed that the curves are more and more flattened as the temperature increases, which means that failure is accompanied by increasingly greater displacements. It is interesting to note that the maximum recorded force had nearly the same displacement value in all four cases (~1 mm).
5 Conclusion

In our work we analyzed the load bearing capacity of anchors placed in thermally damaged reinforced concrete. Our primary goal was to assist the reinforcement work of reinforced concrete structural members damaged in fire events.

In all pull-out tests, concrete cone failure was observed. Among the failures, no specimen showed either a pull-out failure or the combination of concrete cone failure and pull-out failure. This means that the adhesive can create a bond of sufficient quality even in thermally damaged concrete.

The load capacity of anchors created with epoxy adhesive decreased with increasing temperature during thermal loading. When plotting the tensile resistance in function of the temperature, the following conclusions can be drawn:

- if temperature reaches 200 °C in the embedment depth, then tensile resistance drops to 49 %,
- if temperature reaches 300 °C in the embedment depth, then tensile resistance drops to 33 %,
- if temperature reaches 400 °C in the embedment depth, then tensile resistance drops to 32 %.

During the investigation we found no delamination (spalling) of the concrete in any of the specimens, so the results of the test can be used only in cases where spalling does not occur in the reinforced concrete structure during fire.

Change of isotherm lines could be followed in our numerical model. From the isotherms, reduction and distribution of the strength of concrete could be estimated. From the estimated strength distribution, load bearing capacity of bonded anchors also could be calculated. Measured and calculated load bearing capacity values were compared and we found that the calculated values were 1.6 times higher than the measured values, therefore the calculation overestimated the resistance.

The force-displacement curves of pull-out tests performed on the standard specimens describe brittle failure. The initial rapid force uptake, after reaching the maximum load, was followed by a rapid failure at small displacements. In contrary, the force-displacement curves of pull-out tests performed on the thermally stressed specimens show a gradual decrease in tensile resistance. It can also be observed that the curves are more and more flattened as the temperature increases, which means that failure is accompanied by increasingly greater displacements. It is interesting to note that the maximum recorded force had nearly the same displacement value in all four cases (~1 mm).

6 Acknowledgement

The authors wish to thank Szabolcs Kovács-Sebestény and fischer Hungary for providing the necessary anchors.
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INVESTIGATION OF BONDED ANCHORS BY BEAM MODELS IN A DISCRETE ELEMENT FRAMEWORK

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ABSTRACT

Bonded anchors are defined as post-installed fastening systems. Rebars or threaded bars are connected to the base material with a thin layer of adhesive mortar. Reliable models, able to capture the behaviour of the individual materials, are needed to reproduce the system’s behaviour. This contribution shows the calibration and validation of a bonded anchor model under tension loading, based on a discrete element framework. Compressive strength, elastic modulus and fracture energy have been obtained for the concrete at 28 days and have been used to calibrate the concrete model, the Lattice Discrete Particle Model (LDPM)¹. A stress-slip law is used to connect the concrete to the threaded bar, modelled as beam elements. The stress-slip law has been calibrated with so-called confined tests. The validation of the numerical model is performed with comparison between the numerical and experimental response of unconfined tests and failure mode.

1 Introduction

Fastening systems such as post-installed adhesive anchors provide the opportunity to design flexible structures and renovate or strengthen old buildings. The bonded anchors are relatively cost-effective fasteners which are based on a different working principle compared to more traditional mechanical anchors. For the latter, the load is transferred to the concrete at the anchor head in case of undercut or headed stud anchors, and through friction on the anchor surface close to the bottom of the anchor for friction based systems. For bonded anchors, the load is transferred through the thin adhesive mortar layer along the entire bonded length. The different working principle leads to a failure mechanism which is a combination of concrete cone failure and bond failure. Thanks to the steadily improving modelling concepts, numerical modelling has become over time a reliable tool². To describe the behavior of the concrete, many concrete models have been developed during the last years. A reliable and truly predictive numerical model can be a valuable tool that may provide insights into stress state, strain state and damage development. Furthermore, such a model may be useful in order to reduce the number of tests in the laboratory. A numerical model is proposed in this paper based on a discrete element framework. The LDPM is used to simulate concrete and beam elements represent the threaded bar. The interface is modelled through a stress-slip law. The material model parameters are calibrated and validated on a comprehensive dataset which includes concrete characterization and pull-out tests. Compressive strength, tensile strength, fracture energy and elastic modulus are...
obtained experimentally at 28 days of concrete age. In the same day, confined and unconfined pull-out tests are performed on bonded anchors with a vinyl-ester based adhesive mortar. Additionally, a photogrammetric tool is used to acquire the three-dimensional profile of the void left by the concrete cone. Also, the concrete which surrounds the anchor has been cored out after the test to identify the presence of additional cracks along the fastener.

2 Anchor and adhesive mortar layer models

During the experimental campaign definition, the bonded anchors maximum load was design to be within the linear elastic domain of the threaded bar. For this reason, to simulate the threaded bar, a simple elastic material and 1D beam elements with both translational and rotational degrees of freedom have been used. The beam elements are embedded in the concrete domain, no borehole is modelled. The anchor is coupled to the LDPM particles which are around the anchor domain defined by its diameter and its length (Figure 1b). The basic response of the stress-slip law is elastic, with stiffness $K_a$ in the axial direction and $K_r=K_c=p_KK_a$ in the radial and circumferential directions, where $p_K$ is a penalty coefficient. The model assumes stiffness degradation based on a scalar damage variable. If isotropic damage is assumed, all stiffness moduli decrease proportionally and independently of the loading direction. Otherwise, only damage in axial direction is employed. The damage evolution law is postulated in an explicit form, relating the damage variable to the largest previously reached equivalent axial slippage. The pre-peak behaviour is characterised by a power law which connects the elastic branch to the peak, where the curve has a horizontal tangent (Figure 1a). The post-peak behaviour is described by multi-linear softening.

![Figure 1 – Stress-slip law (a) and visual representation of the interaction between interface and particles (b).](image-url)
3 Concrete model

The LDPM is a concrete model defined at the meso-scale (or aggregates scale) which simulates the mechanical interaction of coarse aggregates embedded in a cementitious matrix (mortar). The full description of LDPM is reported in Cusatis et al. The aggregates, which are approximated as spheres, are randomly placed in the concrete domain, following the sieve curve. A lattice mesh is created connecting all the centers of the aggregates. Delaunay tetrahedralization and domain tessellation are performed on the concrete domain to create a system of polyhedral cells which interact with each other through the common triangular facets. In the elastic regime, the normal stress is proportional to the normal strain through a parameter $E_N$, and the shear stresses are proportional to the shear strains through $E_T$ where $E_N=E_0$ ($E_0$=effective normal modulus) and $E_T$ is the product of $E_0$ and $\alpha$ ($\alpha$=shear-normal coupling parameter). Beyond the elastic limit, the LDPM formulation considers: fracture and cohesion, compaction and pore collapse, and friction.

4 Experimental investigation

The data set used for this investigation is part of a larger one used to determine ageing of concrete and fastenings. At this point, only the results at 28 days are used. Tests on concrete are performed to determine its properties, and tests on the system (with different configurations) are performed to obtain data for the stress-slip law calibration and for the model validation.

4.1 Concrete characterization

Standard tests such as cube compression test, cylinder compression test (used also for the determination of the elastic modulus) and three-point bending test have been performed to obtain the concrete properties. The elastic modulus was obtained from the loading branch of a cylinder compression test by recording the deformation with three LVDTs placed in a 120° configuration having 100 mm base length. The cylinder has a diameter of 150 mm and a length of 300 mm. Three of these tests were performed obtaining a result of $22.31 \pm 4.1\%$ GPa. The compressive strength was obtained dividing the peak load of cube compression tests by the cross-section of the specimen having an edge length of 150 mm. Three of these tests were performed obtaining a result of $25.96 \pm 0.8\%$. The fracture energy was obtained by dividing the area under the load displacement curve of a three-point bending test by the fractured area. The specimen was a beam with dimensions $400 \times 100 \times 100$ mm and a notch with 30 mm depth and 4 mm width. Three of these tests were performed obtaining a fracture energy equal to $75.0 \pm 11.2\%$ N/m. Figure 3 shows the experimental results with the mean experimental curve. In the same figure, also the numerical result, which will be explained later on, is plotted.

4.2 Tests on bonded anchors

Pull-out tests have been performed on bonded anchors for all the configurations with an embedment depth of 90 mm and a M12 threaded bar. Two different confinement configurations have been used as shown in Figure 2. In Figure 2a the so called “confined” configuration is shown. The ring used to support the concrete slab has an internal diameter $D_1$ equal to 22 mm and an external one $D_2$ equal to 36 mm. According to the relevant guidelines, for a test to be considered as confined the internal diameter of the ring used as support needs to be 1.5 to 2 times the borehole diameter. Two LVDTs
measuring the relative displacement between grip of the load frame and concrete surface have been used to control the test and record the slippage of the anchor during the test. The confined test will lead to a de-bonding failure, useful for the determination of the bond strength. Assuming a uniform bond stress distribution, the bonded strength is given by the formula:

\[ \tau_c = \frac{N_c}{\pi h_{\text{eff}} d} \]  

where \( N_c \) is the confined pull-out test maximum load, \( d \) is the diameter of the threaded bar and \( h_{\text{eff}} \) is the embedment depth. The calculated bond strength \( \tau_c \) turns out to be equal to 22.15 ± 5.6% MPa. In Figure 2b, the so called “unconfined” set-up is shown. The ring used to support the concrete slab has an internal diameter \( (D_1) \) of 400 mm and an external one \( (D_2) \) of 600 mm. For the test, to be defined as unconfined, the internal diameter of the ring used as support needs to be bigger than four times the anchor embedment depth as suggested in the guidelines. The unconfined configuration will lead to the more complicated combined failure which is composed of a concrete cone failure at the top part of the anchor and a bond failure at the bottom part of the anchor. For the unconfined tests the LVDTs were used both to control the test and to measure the anchor slip. Considering a uniform bond stress distribution, the calculated equivalent bond strength \( \tau_u \) is equal to 15.87 ± 2.1% MPa and was calculated as:

\[ \tau_u = \frac{N_u}{\pi h_{\text{eff}} d} \]  

where \( N_u \) is the unconfined pull-out test maximum load.

Figure 2 - Set-up of confined (a) and unconfined (b) pull-out tests

The experimental results are presented as load displacement curve in Figure 4 along with their mean curve and the numerical results which will be commented on later. To further study the failure mechanism of the bonded anchor two ways have been pursued: the first one through a photogrammetric acquisition of the void left; the second by coring the system after the pull-out test and verifying the presence of cracks along the anchor. The aim is to compare the real concrete cone shape to the concrete cone shape of the numerical simulation to validate the failure mechanism of the numerical model. After coded targets are placed around the area of interest, around 20 pictures have
been taken from different positions and distances, for any of the voids left in the slab after removing the pulled-out concrete cone. The software computes the position of the targets and from them, it determines the position of the camera for every picture taken. After that, the software digitalizes the surfaces and the concrete cone void can be exported as point-cloud. For all the unconfined tests the coordinate system of the point cloud has been transformed from Cartesian to polar in order to obtain an anchor depth-radial distance diagram. Figure 5 shows the individual cone shape of the unconfined tests and their average. In the same figure, also the numerical result, which will be discussed later on, is plot. The concrete surrounding the void left from the concrete cone has been cored out to verify the presence of other cracks along the anchor. Typically, as seen from these specimens, the crack that ends as concrete cone is not the only one which develops. A series of cracks develops along the bar and only one of them will localize.

5 Model Calibration

The calibration starts from the concrete material and then it is performed on the mortar layer. Once the parameters of both the material models are calibrated, the model will be validated on another dataset without modifying any of the parameters.

5.1 Concrete calibration

The concrete properties used for the model calibration are modulus of elasticity $E$, compressive strength $f_c$ and total fracture energy $G_F$. The discrete model used for the concrete material is formulated on the mesoscale\(^1\) and for this reason it needs to be calibrated through inverse analysis.

![Figure 3 - Concrete calibration on (a) cube compression tests and (b) three-point bending tests.](image)

Load-displacement curves need to be reproduced numerically at the same time for all the concrete tests configurations. Figure 3 shows the comparison between the numerical and experimental results for the concrete calibration. The LDPM concrete parameters have been fine tuned to concurrently fit both the experimental results of cube compressive strength and three-point bending tests\(^4\).
5.2 Stress-slip law calibration

For the calibration of the stress-slip law, the model needs to include the concrete slab, bonded to the threaded bar through the adhesive mortar layer.

![Figure 4 - Experimental and numerical results of confined (a) and unconfined (b) pull-out tests.](image)

For this reason, the bond-law parameters need to be calibrated after the concrete calibration is finalized. Also for this part the parameters have been fine-tuned to fit the confined test experimental results. The elastic branch and the peak have been fitted and the post peak has been approximated with a trilinear decay model. The result is shown in Figure 4a.

6 Model Validation

The experimental results available for the validation are: unconfined pull-out tests, digitalized concrete cone shape, crack pattern from cored out concrete cores. Each one of this experimental result will be used for the comparison with the respective numerical results.

6.1 Validation on unconfined configuration

The validation of a model assesses its ability to predict the system response, out of its calibration domain. In this case, the validation is performed on a unconfined pull-out test load displacement curve and experimental evidence concerning the failure mechanisms. Figure 4b shows the numerical result of the unconfined test, plotted on top of the experimental one. The elastic part and the peak is very well reproduced as is the peak load. Also, the trend of the post peak is properly reproduced even though the load is higher for the same slip level. Further improvements can be expected for more refined modelling concepts.
6.2 Validation on failure mechanisms

![Figure 5](image-url) Digitalized concrete cone shapes compared with the numerical concrete cone ones

![Figure 6](image-url) Qualitative comparison between numerical and experimental crack pattern along the anchor

The numerical void, left in the slab by the concrete cone has been also post-processed as the experimental one for direct comparison. Figure 5 shows their comparison where it can be seen that the numerical result lies within the experimental scatter showing good agreement with the experimental results. Figure 6 shows the comparison between the cored and the visual representation.
of the numerical result. From the comparison, it can be seen that for both, multiple cracks are present and in general two main cracks can be identified. Number 1 (in Figure 6 left) is the one which localizes in a concrete cone for both the experimental and numerical results, number 2 is the one which starts at the bottom of the threaded bar and elastically closes after the peak, when the load decreases.

7 Conclusion

Tests performed on concrete and bonded anchors have been used to calibrate the model parameter of the discrete concrete model and of the stress-slip law.

Unconfined tests results have been used to validate the numerical model comparing the load-displacement curve. A novel technique, photogrammetry, has also been used to enrich the experimental result set for model validation. The model shows good predictive capability both in terms of anchor capacity and failure mechanisms.

References:


DETERMINATION OF CRITICAL EDGE DISTANCES FOR SPLITTING FAILURE OF ADHESIVE ANCHORS

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ABSTRACT

Adhesive anchor systems with threaded rods loaded by tension can fail by splitting failure if the member dimensions are relatively small or if the fasteners are installed near an edge or corner. Therefore, the splitting failure mode has to be considered in the design of adhesive anchors in uncracked concrete by using critical edge distances. These distances are derived from tension tests in the corner with an unconfined test setup.

However, due to the wide range of embedment depths (h_{ef} = 4d to 20d) of adhesive anchors and the myriad of combinations of edge distances and member thicknesses, a vast number of tests are required to check all combinations.

A proposal for the numerical derivation of the critical edge distances is presented in order to eliminate the tests. This proposal is based on a design model for the calculation of the splitting failure load and takes into account the different bond capacities of the adhesive systems, concrete properties as well as different member thicknesses.

1 Introduction

In recent years the load capacity of adhesive anchor systems was significantly increased and the range of the embedment depth was enlarged (h_{ef} = 4d to 20d). High performance adhesive anchors with a high load bearing capacity are available and are able to utilize the concrete capacity. On the other hand an increasing number of applications with fastenings installed near an edge or in concrete elements with small member thicknesses appear. Those fastenings may fail by splitting failure and the load may be smaller than the failure load for concrete cone failure or pullout failure. Therefore, the failure mode splitting has to be considered under certain admissible service conditions in the design of these anchors.

It was shown that the splitting failure load of adhesive systems with conventional threaded rods are not affected by the type of the adhesive. Furthermore, a general design method for splitting failure for adhesive anchor systems was proposed. The proposed calculation method conforms to the results of 222 tests with different adhesive anchor systems with threaded rods and it predicts the splitting failure load obtained in the experiments with sufficient accuracy (see Figure 6). Details of the design
method are given in Section 2 to illustrate how installation parameters (edge distances, member thicknesses, embedment depth and material properties) influence the resistance of splitting failure.

Based on this design method, a general approach for the calculation of the critical distances for splitting failure for adhesive anchor systems is derived in order to avoid tension tests in the corner (Section 3).

2 Design method to calculate the splitting failure load

On the basis of more than 220 tests failed by splitting failure with 6 different adhesive anchor systems in 4 different laboratories, a design method was developed. It was shown that the splitting failure load of anchors depends on the dimensions of the application (embedment depth, edge distance, spacing, member thickness) and concrete properties.

In the following, a design method to calculate the splitting failure load for post-installed adhesive anchors is presented.

The mean splitting failure load of an anchor group can be calculated using Equation (1). Equation (1) is based on the average splitting failure load $N_{um,sp}^0$ (Equation (2)) of a single anchor at the edge. The splitting failure load of a single anchor depends on the edge distance, the member thickness and the concrete compressive strength. Moreover, the effect of the load transfer mechanism is taken into account by the factor $k_p$. Please note that this factor is only valid for adhesive anchors with conventional threaded rods. For systems with other load transfer mechanisms (e.g., expansion bonded anchors) different factors apply. The projected areas $A_{csp}$ (Equation (5)) and $A_{0,csp}$ (Equation (6)) take into account the influence of spacing and if applicable of an additional edge effect on the splitting failure load in a uniform manner to the CC-Method for the failure mode concrete edge failure under shear loading.

The ratio $A_{csp}/A_{0,csp}$ predicts that the failure load will be proportional to the projected area and thus directly proportional to the thickness of the concrete member. However, according to experimental investigations the splitting failure load is less than directly proportional to the factor $A_{csp}/A_{0,csp}$. Therefore, the factor $\psi_{h,sp}$ (Equation (7)) is inserted. The effect of the spacing $s_2$ (Figure 1) of anchor groups perpendicular to the edge is considered by the factor $\psi_{s,sp}$ (Equation (9)). It is noted that the critical spacing $s_{cr,sp} = 2c_{cr,sp} = 3c_1$ (Equation (8)) is assumed based on the CC-Method for concrete edge failure. All equations are expressed in SI-units.

\[
N_{um,sp} = N_{um,sp}^0 \cdot \frac{A_{csp}}{A_{0,csp}} \cdot \psi_{h,sp} \cdot \psi_{s,sp} \quad \text{[N]} \quad (1)
\]

with

\[
N_{um,sp}^0 = \text{splitting failure load of a single anchor at the edge, see Figure 3} \quad \text{[N]} \quad (2)
\]

\[
k_p = \text{product factor for unconfined test support (splitting failure) for adhesive anchors} \quad \text{[-]} \quad (3)
\]

\[
k_{p,mean} = 5.93 \cdot d^{0.37} \quad \text{(mean factor)}
\]
\[ d = \text{diameter of threaded rod} \]
\[ c_1 = \text{edge distance, see Figure 1} \quad [\text{mm}] \]
\[ h_{cr,sp} = \text{critical member thickness, see Figure 2} \quad [\text{mm}] \]
\[ h_{cr,sp} = h_{ef} + c_{cr,sp} \quad [\text{mm}] \]
\[ h_{ef} = \text{embedment depth} \quad [\text{mm}] \]
\[ c_{cr,sp} = \text{critical edge distance (splitting failure)} \quad [\text{mm}] \]
\[ c_{cr,sp} = 1,5c_1 \quad (4) \]
\[ f_{cc,150} = \text{concrete compressive strength, measured on cubes 150 mm} \quad [\text{MPa}] \]
\[ A^0 = s_{cr,sp} \cdot h_{cr,sp} \quad \text{for single anchor near the edge, see Figure 3} \quad [\text{mm}^2] \quad (5) \]
\[ A_{esp} = (c_2 + c_{cr,sp}) \cdot h \quad \text{for single anchor near the corner, see Figure 4} \quad [\text{mm}^2] \quad (6) \]
\[ \psi_{h,sp} = \text{factor to account the influence of the actual member thickness} \quad [-] \]
\[ \psi_{h,sp} = \left( \frac{h_{cr,sp}}{h} \right)^{0.5} \geq 1 \quad (7) \]
\[ h = \text{member thickness of the application, see Figure 5} \quad [\text{mm}] \]
\[ h \leq h_{cr,sp} \]
\[ s_{cr,sp} = \text{critical spacing (splitting failure)} \quad [\text{mm}] \]
\[ s_{cr,sp} = 2c_{cr,sp} = 3c_1 \quad (8) \]
\[ \psi_{s,sp} = \left( 1 + \frac{s_2}{s_{cr,sp2}} \right) \leq 2 \quad [\text{mm}] \quad (9) \]
\[ \psi_{s,sp} = \text{factor to account the influence of the spacing } s_2 \quad [-] \]
\[ s_1, s_2 = \text{spacing of anchor groups (see Figure 1)} \quad [\text{mm}] \]

A comparison of the ratio \( N_{u,test} / N_{u,calc} \) (Equation (1)) is shown in Figure 6. A statistic evaluation of 222 tests results in a mean ratio between measured and calculated failure loads \( N_{u,test}/N_{u,calc} = 1,02 \) and COV=18%.

The characteristic splitting failure load \( N_{Rk,sp} \) can be calculated by replacing the factor \( k_p \) (Equation (2)) by the factor \( k_{p,5\%} \) (Equation (10))

\[ k_{p,5\%} = 4.0 \cdot d^{0.37} \quad \text{(characteristic factor)} \quad (10) \]
Figure 2: Definition of the critical member thickness $h_{cr,sp}$

Figure 3: Projected splitting failure area $A_{csp}^0$ of a single anchor at an edge

Figure 4: Projected splitting failure area $A_{csp}$ of a single anchor at a corner with $h \geq h_{cr,sp}$

Figure 5: Projected splitting failure area $A_{csp}$ of a single anchor at a corner in a thin concrete member with $h < h_{cr,sp}$
3 Approach for the calculation of the critical distances

The tension capacity of an adhesive anchor depends on the embedment depth, the bond strength of the system, the diameter of the rod and the concrete properties. With increasing embedment depth the tension capacity is rising. However, as shown in Figure 7 the decisive failure mode under tension loading is changing with rising embedment depth. In Figure 7 the characteristic resistances for pullout (Equation (13), for low ($\tau_{Rk} = 8 \text{ MPa}$); medium ($\tau_{Rk} = 14 \text{ MPa}$) and high ($\tau_{Rk} = 20 \text{ MPa}$) bond strength) and concrete cone failure (Equation (14)) as well as steel failure (calculated for a steel strength $f_{ut} = 1000 \text{ Mpa}$) for sizes M8 (Figure 7a)) and M30 (Figure 7b)) are compared. It is obvious that for small diameters (M8) the tension capacity is limited by steel failure. For larger sizes (M30) the decisive failure mode is pullout or concrete cone failure depending on the embedment depth and the bond capacity. All in all the minimum tension capacity (derived from pullout, concrete cone and steel failure) is achieved at different ratios $h_{ef}/d$. This must be taken into account for the derivation of the critical edge distances.

The corner tests for the determination of the critical edge distance $c_{cr,sp}$ to cover concrete splitting - as required in ETAG001, part 5 (Table 5.5) - presents particular challenges from a testing and assessment standpoint. For a typical test program, the number of anchor diameters (e.g. M8 to M30 and ratios of member thickness to embedment depth sought for recognition result in a large number of tests. Experience has shown that the assessment of these tests can result in widely varying values for the required value of $c_{cr,sp}$ for systems having comparable bond strengths and geometrical
parameters. Research shows, however, that the value of $c_{cr,sp}$ is not dependent on the adhesive anchor system type (epoxy, vinylester, adhesive manufacturer etc.), but rather only on the characteristic bond strength, the embedment depth and the member thickness.

Therefore it was proposed to calculate the critical edge distance $c_{cr,sp}$ for bonded anchors (ETAG: $c_{cr,sp}$) by a simple expression without tests.

According to\(^3, 6\) the requirement for the critical edge distance for splitting failure is fulfilled, if the average failure load in the tests with anchors at the corner is approximately the same as for an anchor without edge and spacing effects for the same concrete strength. This requirement is achieved if Equation (11) is satisfied.

$$
\tau_{Rm,sp} = \min (\tau_{Rm,p}, \tau_{Rm,c})
$$

(11)

with

$$
\tau_{Rm,sp} = \text{Mean bond strength for splitting failure according to Equation (12), factor } k_p
$$

$$
\tau_{Rm,p} = \text{Mean bond strength for pullout failure according to Equation (13)}
$$

$$
\tau_{Rm,c} = \text{Mean bond strength for concrete cone failure according to Equation (14)}
$$

The pullout load capacity of adhesive anchors is determined by the bond strength which is product specific. For (the purpose of) comparison, the bond strength is calculated according to Equation (12) from the mean splitting failure load. The bond strength for splitting failure is compared with the bond strength for pullout failure (Equation (13)) and concrete cone failure (Equation (14)). All equations are expressed in SI-units.

$$
\tau_{Rm,sp} = \frac{N_{um,sp}}{\pi \cdot d \cdot h_{ef}}
$$

(12)

with

$$
N_{um,sp} = \text{Mean splitting failure load according to Equation (1) with factor } k_p \text{ according to Equation (3)}
$$

$$
d = \text{diameter of threaded rod}
$$

$$
h_{ef} = \text{embedment depth}
$$

$$
\tau_{Rm,p} = \frac{\tau_{Rm,p}}{A_{p,N}} A_{p,N} \psi_{s,N}
$$

(13)

with

$$
\tau_{Rm,p} = \text{Average bond strength for adhesive anchor systems with high } (\tau_{um} = 20 \text{ MPa}),
$$

medium (\(\tau_{um} = 14 \text{ MPa}\)) and low (\(\tau_{um} = 8 \text{ MPa}\)) pullout capacity

for $A_{p,N}, A_{p,N}^0, \psi_{s,N}$ see Ref.\(^7\)

$$
\tau_{Rm,c} = \tau_{m,\text{max,uncracked}} = \frac{k}{0.75 - (\pi \cdot d)} h_{ef}^{0.5} \cdot \gamma_{c,ef}^{0.5} \cdot \frac{A_{s,N}}{A_{c,N}} \cdot \psi_{s,N}
$$

(14)
with

\[ \tau_{\text{Rm,c}} = \text{Average bond strength for adhesive anchor systems for concrete cone failure} \]
\[ k = 11 \text{ (Effectiveness factor for uncracked concrete, see Ref.}^7 \text{)} \]
\[ s_{\text{cr},N} = 3h_{\text{ef}} \]

for \( A_{c,N}, A_{o,N}, \psi_{s,N} \) see Ref.\(^7\).

In Figure 8 the measured bond strength of tension tests (converted to \( f_{c,cyl} = 20 \text{ MPa} \)) with size M8 (Figure 8a) and M30 (Figure 8b) at the corner are compared with the calculated mean bond strength for concrete cone failure and the calculated mean splitting failure load according to Equation (1) (C20/25). In the tests with size M8 (Figure 8a) it is shown that despite a member thickness \( h = 2h_{\text{ef}} \) an edge distance of \( c = 1.5h_{\text{ef}} \) it is not sufficient to achieve the required failure load. Besides, it is obvious that the model for the calculation of the splitting failure load agrees well with the test results. According to the requirements\(^3,6\) the necessary splitting failure load is achieved at \( c_{\text{cr,sp}} \approx 2.2h_{\text{ef}} \). This result shows that the current recommendation\(^6\) \( c_{\text{cr,sp}} = h_{\text{ef}} \) for \( h = 2h_{\text{ef}} \) for some applications is incorrect.

The test results with size M30 (\( h_{\text{ef}} = 9d \)) with an edge distance \( c_1 = c_2 = 1.5h_{\text{ef}} \) in thin test members (\( h \approx 1.2h_{\text{ef}} \)) show a 40\% lower splitting failure load compared to the required load (see Figure 8b)). The required edge distance for splitting failure according to the calculations is \( c_{\text{cr,sp}} \approx 2.4h_{\text{ef}} \).

After evaluating a huge number of combinations of different sizes \( 8 \text{ mm} \leq d \leq 30 \text{ mm} \), edge distances, member thicknesses, embedment depth and bond strength it is proposed to calculate the critical edge distances of adhesive anchors for splitting according to Equation (15). The characteristic bond strength \( \tau_{k,\text{uncr}} \) is limited by concrete cone failure (Equation (16)). This approach covers unfavourable combinations.

Please note that Eq. (15) is not valid for torque-controlled adhesive anchors (the model does not apply to these anchors). Furthermore, it is pointed out that an equivalent proposal was already implemented in AC3085 in February 2013.

The proposed approach for the derivation of the required critical edge distances is illustrated in Figure 9 as a function of the ratio of member thickness to embedment depth for low, medium and high bond capacity. The approach shows that with larger bond capacity of the adhesive anchor system the required critical edge distances increases. Furthermore, a larger critical edge distance for splitting for applications with smaller member thickness (\( h/h_{\text{ef}} < 2.4 \)) is necessary.

\[
\begin{align*}
  c_{\text{cr,sp}} &= h_{\text{ef}} \left( \frac{\tau_{k,\text{uncr}}}{8} \right)^{0.4} \cdot \left( 3.1 - 0.7 \frac{h}{h_{\text{ef}}} \right) \\
  &\text{for } 8 \text{ mm} \leq d \leq 30 \text{ mm and } h/h_{\text{ef}} \leq 2.4
\end{align*}
\]

where

\begin{align*}
  h_{\text{ef}} &= \text{embedment depth} \\
  \tau_{k,\text{uncr}} &= \text{characteristic bond strength in uncracked concrete evaluated from tests} \\
  h &= \text{member thickness of the application} \\
  h_{\text{ef}} &= \text{embedment depth}
\end{align*}
\[ \tau_{k, uncr} \leq \frac{k_{uncr} \cdot \sqrt{h_{ef} \cdot f_{ck}}}{\pi \cdot d} \]  

(16)

where

- \( f_{ck} \) = characteristic concrete compressive strength, cylinder
- \( k_{uncr} \) = 11 (uncracked concrete), see Ref.\(^7\)

Representative for the checked combinations the required (derived from tests) and the proposed critical edge distances as a function of the ratio member thickness \( h \) to embedment depth \( h_{ef} \) are shown in Figure 10a) (M8) and b) (M30). As expected, the required edge distances for splitting increase with decreasing member thicknesses. Besides, in Figure 11 the derived critical edge distances of applications with small member thicknesses are compared with the proposed approach (Equation (16)). The diagrams show that the proposal conforms sufficiently with the results of the required edge distances derived from tests for small (\( h_{ef} \leq 6d \), Figure 11a)) and larger (\( h_{ef} \geq 8d \), Figure 11b)) embedment depths.

---

**Figure 7:** Comparison of the characteristic bond strength for pullout \((\tau_{Rk, uncr} = 8/14/20 \text{ MPa})\) and concrete cone failure in uncracked concrete (C20/25) as well as steel failure as a function of the ratio of embedment depth to diameter of the threaded rod.

**Figure 8:** Comparison of the measured bond strength in tests (converted to \( f_{cyl} = 20 \text{ N/mm}^2 \)) with the calculated mean bond strength for concrete cone failure and the calculated mean splitting failure load (Equation (1)), uncracked concrete (C20/25) as a function of the ratio of edge distance to embedment depth.
Figure 9: Ratio of required critical edge distance for splitting $c_{cr,sp}$ to embedment depth $h_{ef}$ to ratio member thickness $h$ to embedment depth $h_{ef}$ according to Equation (15)

Figure 10: Comparison of the required (derived from tests) and the proposed critical edge distances as a function of the ratio member thickness $h$ to embedment depth $h_{ef}$, concrete cone failure decisive

Figure 11: Comparison of the ratio required edge distance to embedment depth derived from tests and according to Equation (15)

a) Size M8

b) Size M30

a) Applications with small embedment depths

b) Applications with large embedment depths
4 Summary

Adhesive anchor systems with threaded rods loaded by tension can fail by splitting failure if the member dimensions are relatively small or if the fasteners are installed near an edge or corner. Therefore, the splitting failure mode has to be considered in the design of adhesive anchors in uncracked concrete by using critical (critical) edge distances\(^1,3\). These distances are derived from tension tests in the corner with an unconfined test setup\(^2,5\).

In the present paper a proposal for the numerical derivation of the critical edge distances for splitting failure is presented in order to eliminate the tension tests in the corner. This proposal is based on a design model for the calculation of the splitting failure load and takes into account the different bond capacities of the adhesive systems, concrete properties as well as different member thicknesses.

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STUDIES ON ANCHORAGE MECHANISM OF INORGANIC-INJECTION TYPE POST-INSTALLED BONDED ANCHOR

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ABSTRACT

Recently, inorganic-injection-type anchors have been developed in Japan. Their consumption amount has been gradually increasing. These inorganic anchors are composed of materials such as cement, aggregate, and additive agents. These components affect the quality and change the bond strength. This study investigated the relationship between material property and bond strength of the inorganic-injection-type post-installed bonded anchor. Furthermore, the anchorage mechanism was estimated, and the ordinary Portland cement (NC) and ultra-rapid-hardening cement (URHC) were used as the main components of the anchor material. This study also investigated the effects of compressive strength, addition of expanding agent, drying shrinkage, and water retention ability on bond strength.

1 Introduction

For bonding-type post-installed anchors, organic post-installed anchors are used frequently. However, from the viewpoint of durability, the use of inorganic post-installed anchors is also increasing.

Generally, in inorganic glass-tube anchors, as the capsules are destroyed and agitated in the hole, the stirring time by the installer is different, and variation tends to occur in the bond strength of the anchor. In the inorganic paper-tube anchors, as the water amount absorbed into the capsule is not constant, the water/cement ratios vary, causing variations in bond strength.

However, in the injection-type anchor, as a specified amount of kneading water is injected into the powder in the cartridge, the water/cement ratio is constant and the material quality is stable. In addition, as the injection amount can be freely changed, it can be applied to those with a large embedded length, and has high flexibility. Therefore, it is speculated that the use applications of the inorganic injection-type post-installed anchors will increase.
For test items and standards of inorganic anchor materials, data that are a clear basis for the standard are not generally shown. Therefore, it is necessary to set test items and standards based on test data of inorganic injection-type post-installed anchors.

2 Experiment

2.1 Purpose

It is necessary to grasp the physical property of anchor material to use them safely. Therefore, we examined the influence of compressive strength, expansion, drying shrinkage, and water-retention ability on bond strength of the anchor using ordinary Portland cement as the base material1,2,3.

2.2 Experiment Outline

The experiments were divided into the following four series.

Series I-: Influence of water/cement ratio and compressive strength

Series II-: Influence of expanding agent

Series III-: Influence of drying shrinkage

Series IV-: Influence of water-retention ability

(1) Materials and Compounding

Table 1 shows the compounding of anchor materials used in this study.

In series I and II, ordinary Portland cement (NC) was used, and the expansion agent was ettringite type. Series III utilized an alkylene-oxide shrinkage-reducing agent to adjust the drying shrinkage by using ordinary NC and ultra-rapid-hardening cement (URHC). Series IV uses NC and URHC, and a methyl-cellulose thickener was used to adjust the water-retention ability.

The water/cement ratio of Series I ranged among 35%, 40%, and 45%, and that of the other series was fixed to 35%. The compressive strength of the base concrete was approximately 45 N/mm², and the dimensions of the base concrete were 800 × 1700 × 400 mm. The anchors were installed onto the base concrete three or more months after its casting. As an anchor, a high-strength reinforcing bar of D13 (MK785) was used, and the tip shape was cut off.

The compressive strength of the anchor material was measured according to JIS A 11084 and JIS A 11495 by using a columnar specimen of φ 50 × 100 mm. The curing was performed under the same conditions as that of the bond strength test specimen, and the measurement was simultaneously conducted using the bond strength test.
Table 1: Compounding of anchor materials

<table>
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<tr>
<th>NC Type</th>
<th>w/c (Water/Cement) (%)</th>
<th>Unit amount compounding (kg/m³)</th>
<th>Unit amount compounding (kg/m³)</th>
<th>Unit amount compounding (kg/m³)</th>
<th>Unit amount compounding (kg/m³)</th>
<th>Unit amount compounding (kg/m³)</th>
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<td>938</td>
<td>-</td>
<td>935</td>
<td>0</td>
</tr>
<tr>
<td>URHC-2</td>
<td>35</td>
<td>326</td>
<td>-</td>
<td>934</td>
<td>-</td>
<td>931</td>
<td>9</td>
</tr>
<tr>
<td>URHC-3</td>
<td>35</td>
<td>324</td>
<td>-</td>
<td>930</td>
<td>-</td>
<td>927</td>
<td>19</td>
</tr>
<tr>
<td>URHC-4</td>
<td>35</td>
<td>322</td>
<td>-</td>
<td>922</td>
<td>-</td>
<td>919</td>
<td>37</td>
</tr>
<tr>
<td>URHC-5</td>
<td>35</td>
<td>327</td>
<td>-</td>
<td>938</td>
<td>-</td>
<td>935</td>
<td>-</td>
</tr>
<tr>
<td>URHC-6</td>
<td>35</td>
<td>327</td>
<td>-</td>
<td>937</td>
<td>-</td>
<td>934</td>
<td>3</td>
</tr>
</tbody>
</table>

Shrinkage-reducing agent: alkylene oxide
Thickener: cellulose

The drying shrinkage was measured according to JIS A 1129-3⁶. The specimen was molded to 40 × 40 × 160 mm, and curing was performed at 20 °C and 60% RH.

The water-retention ability of the anchor material was investigated in the fresh state according to JIS A 6916-2014⁷. In the test, a filter paper with a diameter of 185 mm, as prescribed in JIS P 3801 (filter paper for chemical analysis), was placed on a glass plate and a ring frame with an inner diameter of 50 mm and 10 mm height was installed in the center. The sample was filled in a ring form, scooped up with a spatula, and a glass plate was placed on it. Next, the sample was sandwiched, and allowed to stand upside down. After 60 min, the major and minor diameters of the moisture oozing out into the filter paper were measured using a caliper, the average value \( L_{60} \) (mm) was determined, and the water-retention ability was calculated as follows (Figure 1):

\[
R_w = \frac{L_r}{L_{60}} \times 100
\]  

(1)

where \( R_w \) is the water-retention ability, \( L_{60} \) is the moisture size after 60 min, and \( L_r \) is the inner diameter of the ring form.

(2) Anchor Installation

A diameter of 18 mm was drilled into the base concrete by using a wet core drill, and the buried length of the anchor was set to 7dₐ. Anchors were installed the day after the drilling in Series I–III, and 14 days after the holes were dried in Series IV. The anchor material was kneaded using the
mixing equipment specified in JIS R 5201 for 3 min, and then the hole was filled with the material. After filling the hole, the anchor was immediately inserted, and fixed. Next, the curing was conducted indoors for 14 days.

(3) Bond strength test of anchors

After the curing, the anchor-pullout bond strength was tested using the test apparatus shown in Figure 2. A reaction plate (thickness: 22 mm, hole diameter: 26 mm) was used to make the failure mode a bonding failure. Regarding the force, a stressing chair, center-hole hydraulic jack, and spherical seat were set on the reaction plate, and tensile force was introduced to the anchor by the center-hole hydraulic jack. The measurement items were the load and displacement length of anchor. The bond strength of anchor was calculated as follows:

$$\tau_b = \frac{P_{\text{max}}}{\pi \cdot d_a \cdot l_b}$$

where $\tau_b$ is the bond strength of anchor (N/mm²), $P_{\text{max}}$ is the maximum tensile load (kN), $d_a$ is the anchor nominal diameter (mm), and $l_b$ is the embedded length of anchor (mm), which was obtained by subtracting the length of the non-embedded anchor from the entire anchor length.

### 2.3 Experimental Results

Table 2 shows the average value of five tests in all the series. The bond strength was obtained as 7.1-21.1 N/mm² and greatly varied depending on the compounding.

(1) Series I-: Influence of water/cement ratio and compressive strength

Figure 3 shows the relationship between compressive and bond strengths of the anchor material. When only the water/cement ratio increased, the bond strength tended to decrease with the compressive strength of the anchor material.

When comparing the influence of only the water/cement ratio, a relationship was observed between the compressive and bond strengths of the anchor material; in contrast, the same correlation was not found in all the tests. This is probably because factors other than the material strength largely contribute to the bond strength.
Figure 4 shows the relationship between the failure area and bond strength. The tendency of the failure area at the interface between concrete and anchor material to be larger lower s the bond strength. Moreover, the interfacial bond strength between the concrete and anchor material is determined to largely influence the maximum bond strength of the inorganic-injection-type post-installed anchor. In addition, the bond strength was inferred to be determined by the ratio of failure area to the bond strength in the low bond-strength region (20 N/mm² or less). Photo 1 shows the failure state of the anchor bonding part.

Table 2: Result of bond strength test

<table>
<thead>
<tr>
<th>Term (holing anchor installation) (days)</th>
<th>Bond strength</th>
<th>Compressive strength of anchor material (N/mm²)</th>
<th>Failure Area</th>
<th>Water retention ability (%)</th>
<th>Length change rate (28 d) ((×10^{-6}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC Type</td>
<td></td>
<td>Compressive strength of concrete (N/mm²)</td>
<td>Bond strength</td>
<td>Concrete - Anchor material</td>
<td>Anchor Material - Anchor</td>
</tr>
<tr>
<td>NC-1</td>
<td>1</td>
<td>13.7 21.1 62.7 50 50 - -</td>
<td>46.2</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
</tr>
<tr>
<td>NC-2</td>
<td></td>
<td>7.1 4.5 49.5 30 70 - -</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NC-3</td>
<td></td>
<td>18.3 16.1 55.0 10 90 - -</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NC-4</td>
<td></td>
<td>17.6 18.1 18.5 30 70 - -</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NC-5</td>
<td></td>
<td>14.0 42.5 56.0 60 40 -1040</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NC-6</td>
<td></td>
<td>10.4 42.5 48.1 70 30 -710</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NC-7</td>
<td></td>
<td>8.4 22.2 50.1 80 20 -610</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NC-8</td>
<td></td>
<td>8.8 15.5 47.7 80 20 -470</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NC-9</td>
<td></td>
<td>8.1 44.9 62.6 70 30 35 -</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NC-10</td>
<td>14</td>
<td>10.5 37.2 54.2 60 40 63 -</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NC-11</td>
<td></td>
<td>16.0 18.3 54.3 40 60 83 -</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NC-12</td>
<td></td>
<td>19.5 3.9 49.9 20 80 91 -</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>NC-13</td>
<td></td>
<td>20.4 4.7 44.2 0 100 -860</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>URHC Type</td>
<td></td>
<td>21.1 4.0 47.5 0 100 -500</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>URHC-1</td>
<td></td>
<td>20.3 9.2 45.6 0 100 -490</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>URHC-2</td>
<td></td>
<td>20.1 7.0 42.9 0 100 -470</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>URHC-3</td>
<td></td>
<td>10.9 37.4 47.1 50 50 41 -</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>URHC-4</td>
<td></td>
<td>19.7 4.8 40.2 0 100 85 -</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>URHC-5</td>
<td></td>
<td>14</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
<tr>
<td>URHC-6</td>
<td></td>
<td>25</td>
<td>10.6 13.7 58.6 40 60 - -</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3: Compressive strength of anchor material and bond strength

Figure 4: Failure area (Concrete - Anchor material) and bond strength
(2) Series II: Influence of expanding agent

Figure 5 shows the relationship between the amount of the expanding agent, compressive strength and bond strength. It was confirmed that when the addition amount of the expanding agent was approximately 100 kg/m³, the bond strength increased. When a further expanding agent was added, the bond strength was not increased, and the compressive strength decreased because of excessive expansion of the anchor material itself.

![Figure 5: Amount of expanding agent and compressive strength, bond strength](image)

(3) Series III: Influence of drying shrinkage

Figure 6 shows the relationship between drying shrinkage and bond strength. In both NC and URHC, no relation between drying shrinkage and bond strength was observed.

In the series using NC, the dispersion of the bond strength tended to decrease with the addition of the shrinkage-reducing agent. Owing to the small variation in bond strength, the shrinkage-reducing agent acts as a surfactant so that its addition causes the wettability at the interface of concrete and anchor material to become favorable, and the reaction becomes more uniform. Moreover, the variation is considered to decrease.
Figure 6: Drying shrinkage and bond strength

(4) Series IV : Influence of water-retention ability

Figure 7 shows the relationship between water-retention ability and bond strength. In both the NC and URHC, bond strength tends to increase with the water-retention ability. The increase in water-retention ability leads the failure area to change.

The basic average bond strength$^1$ obtained from the compressive strength of base concrete is 14.6 N/mm$^2$ ($\tau_{bavg} = 10 \sqrt{\sigma_B/21}$). With NC, the water-retention ability was 80% or more, and with URHC, the ability was 65% or more, the bond strength was above its basic average.

For the compounding of NC-10 and NC-12, the voids and fragile layers at the interface were investigated. A concrete test specimen of φ100 × 200 was used, and the anchor was installed after drying for 14 days post drilling. The specimen was sliced at every 30 mm thickness and impregnated with a resin containing a fluorescent pigment. After the resin was cured, the concrete surface was polished to remove excess resin. Photo 2 and 3 show the results of observing the cut surface.

In NC-10 comprising small water-retention ability, many voids were observed at the interface of the anchor material and concrete. In NC-12 comprising high water-retention ability, many voids were observed in the entire anchor material. These observations confirm that the fragile parts were formed at the interface of the anchor material and concrete because of the occurrence of the dry-out phenomenon, when the water-retention ability was low.

Figure 7: Water-retention ability and bond strength
Photo 2: Interface of the anchor material and concrete (NC-10 water-retention ability: 35%)

Photo 3: Interface of the anchor material and concrete (NC-12 water-retention ability: 83%)
3 Estimation of Anchorage Mechanism in Inorganic Post-Installed Anchor

We estimated the anchorage mechanism in inorganic post-installed anchor from the results derived in section 2.

The water-retention ability and the addition of expansion agent greatly affected the bond strength, and the compressive strength and shrinkage performance showed no significant influence. In addition, the bond strength at the interface of concrete and anchor material was determined to be greatly affected. Thus, when reliable bond strength is obtained at the interface of concrete and anchor material, failure occurs at the interface of anchor and anchor material; therefore, it was speculated that during the anchor-pullout, the influence of the compressive strength of the anchor material is observed on the maximum tensile load ($P_{\text{max}}$).

It is also conceivable that the wettability of the concrete surface greatly changes because of the influence of the shrinkage-reducing agent as the surfactant, as shown in the study by Hamanaka et al.\textsuperscript{10}. When the anchor material has higher water-retention ability, the bond strength improves, as reported by Sakakibara et al.\textsuperscript{3}.

Regarding the influence of the expanding agent, the bonding condition is considered to improve by the expansion force at the interface of concrete and anchor material because of the expansion action of the anchor material.

Based on these results, Figure 8 shows a conceptual model that estimates the mechanism of bonding condition of cement anchors.

(A): Anchor materials with high water-retention ability

As the cement material enters the pores of the concrete and moisture is not deprived to the concrete side, cement hydration progresses and dense cement-hydrated crystals are produced. In the anchorage mechanism, a bonding force is generated through mechanical interlocking and intermolecular force.

(B): Anchor materials with low water-retention ability

Even if the cement material enters the pores of the concrete, moisture is deprived to the concrete side; thus, there is little cement hydration resulting in a sparse hydrated crystal, and a fragile layer is formed at the concrete interface. Owing to the fragile layer generated at the interface, the material strength of the mechanically interlocked part and intermolecular force are low.

(C) Anchor materials containing expansion agent with low water-retention ability

Although a fragile layer is formed at the interface on the concrete, the fragile layer thins because of material expansion, thereby increasing the anchor bond strength. Although a bonding force is caused by the mechanical interlocking at the interface due to expansion of the material, the intermolecular force is low.
Figure 8: Conceptual model of bonding in inorganic anchor

(A): Anchor materials with high water-retention ability

(B): Anchor materials with low water-retention ability

(C): Anchor materials containing expansion agent with low water-retention ability

Concrete

Anchor material

Mechanical interlocking and bonding by intermolecular force

Dense

Sparse

Cement hydrate: Crystal state [11]
4 Proposal of Evaluation Criteria for Inorganic Post-Installed Bonded Anchor

Earlier studies have not provided a clear evaluation criterion for inorganic post-installed anchors; thus, it is necessary to propose an evaluation standard of a suitable material to secure a reliable bonding property.

Section 2 showed that water-retention ability is necessary for the performance of the inorganic anchors, and anchors possessing high water-retention ability can obtain sufficient bond strength at concrete and anchor material interface.

The bonding test results on the water-retention ability showed that the water-retention ability of the anchor material that is higher than the standard average bond strength is 80% or more when using either NC or URHC.

In addition, the material strength is considered as the performance necessary for the inorganic anchor. Since compressive force acts on the anchor ribs, it is considered that a reference value should be set for compressive strength.

Figure 9 shows the relationship between compression strength of anchor material, water-retention ability and bond strength. In materials with high water-retention ability and expandability, bond strength tended to be high.

Based on the test data obtained in this study, we propose the evaluation standard value of inorganic post-installed anchor material in Table 4.

![Figure 9: Compressive strength and bond strength of anchor material](image)

<table>
<thead>
<tr>
<th>Water-retention ability : over 80%</th>
</tr>
</thead>
</table>

Table 4: Proposal of evaluation criteria for inorganic anchor materials

<table>
<thead>
<tr>
<th></th>
<th>Reference value</th>
<th>Test method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength</td>
<td>More than 45N/mm²</td>
<td>JSCE-G552</td>
</tr>
<tr>
<td>Water-retention ability</td>
<td>More than 80%</td>
<td>JIS A 6916</td>
</tr>
</tbody>
</table>
5 Conclusion

In this study, to elucidate the anchorage mechanism of inorganic post-installed anchor, we investigated the relation between the bond strength and anchor material properties, and investigated the main factors affecting the bond strength. These results showed that the anchorage mechanism of inorganic anchors and proposed evaluation criteria is necessary for inorganic anchor materials.

1) It was revealed that bond strength greatly decreases because of the fragile layer formed at the interface of concrete and anchor material with bond strength of 20 N/mm² or less.

2) The existence of a correlation between water-retention ability and bond strength of anchor material was clarified. To avoid formation of a fragile layer at the concrete interface and anchor material, it is necessary to improve the water-retention ability of the material.

3) By providing an anchor material with expansion performance, it was shown that its bond strength is higher than the material without expansion performance because of the effect of reducing the fragile layer, even if the fragile layer is formed at the interface of concrete and anchor material.

4) A conceptual model of the force transmission mechanism against the tensile force due to the presence or absence of dry-out was developed for showing the anchorage mechanism of inorganic anchors.

5) The performance required for the inorganic post-installed anchor material was shown, and compressive strength and water-retention ability were proposed as the evaluation criteria.

References:

7. Japan Industrial Standard: JIS A 6916 (ground adjustment material for construction), 2014
9. Civil Engineering Society: Concrete Standard Specifications (Design), Established 2012
IN-SITU QUALITY ASSESSMENT TESTS ON POST-INSTALLED ADHESIVE ANCHORS IN AN EXISTING BUILDING

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ABSTRACT

The performance of post-installed adhesive anchors is really influenced by the quality of operation. Therefore, an appropriate management of operation method is very important.

This report is the results of bond strength test, which is in-situ quality assessment tests on post-installed adhesive anchors applied for three directions, upward, downward and horizontal in the existing building. These tests were conducted in the existing building. The content of the investigation is the hole diameter, embedment depth, anchor bolt angle, and bond strength.

Many reports adopt about 5 specimens per test series. This time, the number of anchor specimens were 65 in each series. They were installed by a single worker.

The test results showed about the same bond strength for upward and horizontal. In case of downward application, the bond strength was relatively low. But they satisfy the design bond strength of AIJ Design Recommendation for composite construction. The difference of bond strength is influenced by the condition of concrete.

Some of the post-installed adhesive anchors were cored off to confirm filling condition. It was confirmed that adhesive was well injected in every directions. The difference in the setting direction has little effect on injection and bond strength. The bond strength in the existing building showed about 70 to 80% values compared with laboratory results.

1 Introduction

In Japan, post-installed anchors have not been approved for structural applications except for seismic reinforcing. Adhesive anchors are mainly used for seismic reinforcing within column/beam framework. Because of its short embedment length, 8 to 12 times of anchor diameter, capsule type adhesive anchors are widely used. In the future, post-installed anchors are expected to be used for connection of existing and new reinforcing applications. The background social needs are effective usage and rationalization of existing buildings.
In case of the expansion of application limit, the embedment length of anchors are expected to be longer than before, and capsule-type adhesive anchors will have difficulty in proper setting operation. Such being the reason, here in Japan, we are trying to adopt injection type adhesive anchors capable for longer embedment length, in order to expand application limit.

The performance of injection type anchors are quite influenced by operation quality. Therefore in order to assure the anchor performance, well-trained anchor installers with enough knowledge of products and operation are necessary, and anchor setting operation needs to be carried out with checking the quality control items in each setting stage by anchor installers, general contractors and designers.

Many reports concerning drilling and setting operations of adhesive anchors adopt about 5 specimens per parameter level of concrete base materials. The objective of this report is operational accuracy and bond strength scattering installed in different directions (upward, horizontal, downward) by a single worker.

2 Outline of study

We took measurements of drilling depth for drilling and anchor setting operation, drilled hole diameter, embedded length and inclination angle of anchor bolt and bond strength for each setting direction (upward, horizontal, downward). After bond strength test, some of the anchor bolts were cored off and injection condition of adhesive was confirmed.

2.1 Parameters and levels

The parameters and level of anchor setting are shown Table 1. The number of anchor specimens is 65 per setting direction, amounting to 195 specimens for 3 directions. We spent one day for drilling operation and next one day for anchor setting operation. The minimum air temperature was 12°C. Since the ceiling height was 2.3m, we used a temporary scaffolding for upward operation to the ceiling.

<table>
<thead>
<tr>
<th>Test parameters</th>
<th>Number of level</th>
<th>Test contents</th>
</tr>
</thead>
<tbody>
<tr>
<td>Setting direction</td>
<td>3</td>
<td>upward(ceiling),horizontal(wall),downward(floor)</td>
</tr>
<tr>
<td>Number of specimens</td>
<td>1</td>
<td>65 for each direction, total 195</td>
</tr>
<tr>
<td>Anchor bolt and diameter</td>
<td>1</td>
<td>SD 785, D13</td>
</tr>
<tr>
<td>Drilling diameter</td>
<td>1</td>
<td>16mm</td>
</tr>
<tr>
<td>Embedment length</td>
<td>1</td>
<td>5da (5×13=65mm)</td>
</tr>
<tr>
<td>Adhesive injection method</td>
<td>1</td>
<td>Cartridge type injection system</td>
</tr>
<tr>
<td>Drilling method</td>
<td>1</td>
<td>Hammer drill</td>
</tr>
<tr>
<td>Anchor installer</td>
<td>1</td>
<td>One single worker with JCAA certificate</td>
</tr>
</tbody>
</table>
2.2 Materials used
Anchor bolt made of high tension reber SD785 was used to avoid yielding. Adhesive is organic epoxy resin was shown in Figure 1.

2.3 Target building
The existing building used for this test is a 5-storied apartment house of reinforced concrete structure built in 1967. This building is vacant at the moment. The anchors were installed in indoor RC members load-bearing wall with 180mm thickness and slab with 110mm thickness. Table 2, Figure 2 and 3 show outline of building, plan of standard floor and outside appearance of this building respectively.

Table 2: Outline of building

<table>
<thead>
<tr>
<th>Location</th>
<th>Kiyose City, Tokyo</th>
</tr>
</thead>
<tbody>
<tr>
<td>Apprication of building</td>
<td>Apartment house (vacant now)</td>
</tr>
<tr>
<td>Date of construction</td>
<td>1967</td>
</tr>
<tr>
<td>Structure type, number of floors</td>
<td>Wall-type reinforced concrete, 5stories</td>
</tr>
<tr>
<td>Design compressive strength of concrete</td>
<td>Fc=165kg/cm² (16.1N/mm²)</td>
</tr>
</tbody>
</table>

Figure 1: Injection systems

Figure 2: Building configuration (room arrangement)

Figure 3: Outside appearance of building
3 Operation process

The operation process is shown by ①~⑦, and installation state in Figure 4 and 5.

3.1 Operation process

①【Rebar investigation】Detection of embedded rebars with rebar detector

②【Marking of drill positions】Marking of drilling positions considering rebar location and anchor spacing

③【Drilling】After drilling depth marking on drill bits, drilling up to the marking position

④【Hole cleaning】Cleaning with vacuum cleaner, brush etc.

⑤【Adhesive injection】Injection tube with filling position marking is inserted in the hole in advance, and adhesive is injected with dispenser.

⑥【Anchor bolt embedment】Anchor bolt shall be embedded up to the hole bottom, and confirmation of adhesive overflow is to be conducted.

⑦【Position fixing and drop prevention】Anchor bolts fixed to upward direction shall be wrapped with rubber band for position fixing and drop prevention.

4 Bond strength test method

4.1 Inspection items

In order to confirm the accuracy of anchor bolts, the items in Table 3 were measured for all anchor specimens after drilling as well as anchor installation process. The embedded length of anchor was obtained by deduction of 4 protruding length from 3 total length of anchor bolt in this Table. This
embedded length values were adopted for calculation of bond strength from tensile test results. The test condition of tensile test of anchors installed on the ceiling is shown in Figure 5.

Table 3: Inspection items

<table>
<thead>
<tr>
<th>Timing</th>
<th>Items</th>
<th>Confirmation method</th>
</tr>
</thead>
<tbody>
<tr>
<td>After drilling</td>
<td>1. Drilled hole diameter</td>
<td>Caliper</td>
</tr>
<tr>
<td></td>
<td>2. Drilled depth</td>
<td>Tape measure</td>
</tr>
<tr>
<td></td>
<td>3. Total length of anchor bolt</td>
<td>Tape measure</td>
</tr>
<tr>
<td>After installation</td>
<td>4. Protruding length of anchor bolt</td>
<td>Tape measure</td>
</tr>
<tr>
<td></td>
<td>5. Inclination angle of anchor bolt</td>
<td>Digital angle measure</td>
</tr>
<tr>
<td></td>
<td>6. Bond strength test</td>
<td>Tension tester</td>
</tr>
<tr>
<td></td>
<td>7. Injection status of adhesive</td>
<td>Breakage of core samples</td>
</tr>
</tbody>
</table>

4.2 Bond strength test

We used a tensile tester with specification as shown in Figure 6, which is available on the market, and measured maximum loads. Since the aim of this test is bond strength, confined test was conducted to prevent concrete cone breakage.

![Out of tester](image1)

Figure 6: Out of tester

5 Test results

5.1 Operation quality

After installation work using a hammer drill with a drill bit of 16mm diameter (measured diameter 16.1mm), we measured drilled diameter, embedded length and inclination angle. Table 4 gives mean, maximum and minimum values and Figure 7~9 give frequency distribution tables.
Drilled hole diameter

The tendency is that drilled diameter is different in the order of wall>ceiling>slab, but the difference is very small. The mean value is about 16.1mm, the minimum is 15.84mm at slab and the maximum is 16.40mm at wall. Thus the operation quality is quite high. Some of the measured values are smaller than drill bit diameter, and the reason may be rough surface of holes causing measurement error.

Drilled depth

Drilling depth is slightly different depending on hole location, and we investigated whether the design value of 65mm is satisfied, and confirmed that all holes had enough depth beyond 65mm.

Embedment length

The tendency is that the scattering of embedment length is big in the order of ceiling>wall>slab. While some of the embedment length on ceiling was long enough, 4 out of 65 anchors gave smaller values than design value 65mm (5da). Thus operation quality has some problem. Since the drilling depth exceeds 65mm, some displacement of anchor bolt is considered after anchor installation, and another study of position fixing will be necessary when setting to upward direction. On the other hand, all anchors to wall and floor gave satisfactory result beyond design value.

Inclination angle of anchor bolts

The tendency is that inclination is large in the order of wall>ceiling>slab. But when setting to wall and ceiling, the operator seemed to set anchors with inclination angle intentionally. In case of horizontal direction, the operator had tendency to set anchor bolts with slant position so that anchor bolts touch the lower surface of hole entrance in order to avoid drop of anchor bolts during hardening process and production of vacant clearance in the holes. (Figure 10) Also when setting anchors to upward direction, adjacent anchors are tied up together with rubber band in order to secure position fixing due to contact between anchor bolt and hole entrance. Thus some inclination angle is automatically produced. When drilled perpendicular to work surface, the maximum inclination angle is around 2.6° based on calculation as shown in Figure 10. Therefore bigger inclination angle over this theoretical value indicates that slant drilling and measuring error are added. Inclination angle of anchor bolts was measured in two directions (X-axis, Y-axis) and the scatter distribution amounts to 130 data per measuring point.

Filling condition of adhesive

We confirmed the filling condition of adhesive by taking out concrete core samples around anchor bolts (7, 6 and 9 pcs from wall, ceiling and floor respectively). In all cases, adhesive was enough filled in holes. An example of this confirmation at ceiling is shown in Figure 11.
Yasunari Fujii, Kiyoshi Imai, Tomoaki Akiyama and Takuya Numata

Table 4: Drilled hole diameter, Embedment length, Inclination angle of anchors

<table>
<thead>
<tr>
<th>Setting direction</th>
<th>Drilled hole diameter (mm)</th>
<th>Embedment length(mm)</th>
<th>Inclination angle of anchors $R(\degree)$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>The mean value</td>
<td>Max</td>
<td>Min</td>
</tr>
<tr>
<td>Wall</td>
<td>16.16</td>
<td>16.40</td>
<td>15.93</td>
</tr>
<tr>
<td>Ceiling</td>
<td>16.10</td>
<td>16.31</td>
<td>15.90</td>
</tr>
<tr>
<td>Slab</td>
<td>16.04</td>
<td>1.35</td>
<td>15.84</td>
</tr>
</tbody>
</table>

Figure 7: The frequency distribution table of drilled hole diameter

Figure 8: The frequency distribution table of embedment length
5.2 Result of bond strength test

Table 5 gives the mean, maximum, minimum values, standard deviation as well as 95% reliability value (degree of freedom 64(N-1) of t-distribution chart, and one side 95% coefficient m=1.669 were adopted). And bond strength distribution is shown in Figure 12. The mean values of bond strength were nearly the same 26.0 N/mm² for both wall and ceiling and 23.2 N/mm² for floor. Concrete strength of structural members range between 42~48 N/mm² as shown in Table 6, and no big difference is observed. Coefficient of variation was about the same 12% for wall and floor and lowest value 9% for ceiling. Even though bond strength for floor is relatively small and the coefficient of variation is high, the lowest value of bond strength is 15.8 N/mm² and this satisfies the design value $10\sqrt{(\sigma_B/21)}$ N/mm² that is commonly adopted.
It seems that the reason of low bond strength of floor does not come from setting direction but from surface condition of structural member. Laitance is not fully eliminated from the surface layer, and fragile layer has been recognized. (Figure 13)

At the fragile area of surface layer, potential bond strength seems to have not been attained. On the other hand, high bond strength and small coefficient of variation at ceiling seems to have been obtained because that surface layer consists of sound concrete due to formwork surface and lower side of structural member, and also good concrete confinement with shear plate as is mentioned later. As for soundness degree of structural members used in this test, condition of ceiling was most stable among wall, ceiling and floor. As is mentioned above, mean value of bond strength is nearly the same for ceiling and wall and low bond strength of ceiling seems to be derived from bad soundness of surface layer of structural members, we come up with the conclusion that the influence of setting directions (upward, horizontal, downward) on bond strength is not so big in existing jobsite operations.

Table 5: Bond strength

<table>
<thead>
<tr>
<th>Setting direction</th>
<th>Bond strength(N/mm²)</th>
<th>Coefficient of variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>The mean value</td>
<td>Max</td>
</tr>
<tr>
<td>Wall</td>
<td>26.5</td>
<td>33.1</td>
</tr>
<tr>
<td>Ceiling</td>
<td>26.8</td>
<td>31.0</td>
</tr>
<tr>
<td>Slab</td>
<td>23.2</td>
<td>29.2</td>
</tr>
</tbody>
</table>

Figure 12: Bond strength distribution

Table 6: Concrete strength of structural members and coefficient of validation
<table>
<thead>
<tr>
<th>Structural members</th>
<th>Concrete strength $\sigma_B$ (N/mm$^2$)</th>
<th>Bond strength (N/mm$^2$)</th>
<th>Ratio based on specimen concrete</th>
<th>Coefficient of variation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>The mean value</td>
<td>$\tau / \sqrt{(\sigma_B/21)}$</td>
<td></td>
</tr>
<tr>
<td>Wall</td>
<td>42.3</td>
<td>26.5 (0.84)</td>
<td>18.7 (0.75)</td>
<td>12.3</td>
</tr>
<tr>
<td>Ceiling</td>
<td>26.8</td>
<td>26.8 (0.85)</td>
<td>17.6 (0.71)</td>
<td>9.1</td>
</tr>
<tr>
<td>Slab</td>
<td>23.2</td>
<td>23.2 (0.73)</td>
<td>16.2 (0.65)</td>
<td>12.8</td>
</tr>
<tr>
<td>Specimen concrete</td>
<td>33.7</td>
<td>31.5 (1.00)</td>
<td>24.9 (1.00)</td>
<td>5.4</td>
</tr>
</tbody>
</table>

### 6 Review

Operation quality

Since the purpose of this test is confirmation of bond strength, we carried out confined tests by setting shear plate around hole entrance and expected all failure mode was bond failure. But actually uneven surface layer caused such test condition that shear plate did not confine completely. Also confinement with shear plate sometimes did not produce bond failure because of fragile surface layer of structural members. Such being the situation, some of the specimens showed slight delamination of surface layer of concrete (Figure 14 surface layer delamination), or cone failure of surface layer concrete with about 10mm depth (Figure 15 surface layer cone). Table 7 shows the relationship between failure mode difference of surface layer and bond strength. When other failure mode than bond failure does not exist, we call this “no failure”. In all structural members, bond strength gets smaller in the order of “no failure”>”surface layer delamination”>”surface layer cone”, and in case of surface layer cone, the bond strength decreased down to about 90%. As for the case of wall, the surface of structural member was sound enough, and uneven surface seems to be the reason of surface layer delamination and surface layer cone. In case of floor, existence of fragile surface layer seems to have caused surface layer delamination and surface layer cone. When carrying out bond strength test at existing buildings, uneven surface sometimes exists compared with concrete specimens, and special attention is necessary to adjust the uneven surface in order not to spoil the confinement effect of shear plate.
Comparison of bond strength with laboratory test

We conducted potential bond strength test at our laboratory using the same epoxy resin. Its result is shown in Table 6. Jobsite test gives the bond strength 23.2~26.8 N/mm² compared to laboratory result 31.5 N/mm² on an average. That means 70~80% of potential values are available. Considering the difference of concrete strength we adjusted the jobsite result with compensation coefficient $\sqrt{\sigma_B/21}$, and even in this case more than 65% of potential strength is available. (Concrete strength beyond 36 N/mm² is out of application limit and just for your reference) As for coefficient of variation, about 10% is observed on jobsite compared to stable value 5.4% at laboratory test. This difference is the result of scattering of concrete quality, and difference of operating condition, atmosphere and confinement condition.

Because there are many atmosphere and operating conditions, future accumulation of back-up data is necessary to get bond strength situation at existing buildings considering operation quality of post-installed anchors.

![Figure 14: Surface layer delamination](image1)

![Figure 15: Surface layer cone](image2)

Table 7: Relationship between failure mode difference of surface layer and bond strength

<table>
<thead>
<tr>
<th>Structural members</th>
<th>Failure mode of surface layer</th>
<th>Number of anchor bolt</th>
<th>Bond strength (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>The measure value</td>
</tr>
<tr>
<td>Wall</td>
<td>Not failure</td>
<td>27</td>
<td>28.0</td>
</tr>
<tr>
<td></td>
<td>Surface layer delamination</td>
<td>18</td>
<td>26.0</td>
</tr>
<tr>
<td></td>
<td>Surface layer cone</td>
<td>20</td>
<td>24.8</td>
</tr>
<tr>
<td>Ceiling</td>
<td>Not failure</td>
<td>64</td>
<td>26.9</td>
</tr>
<tr>
<td></td>
<td>Surface layer delamination</td>
<td>1</td>
<td>19.3</td>
</tr>
<tr>
<td></td>
<td>Surface layer cone</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Slab</td>
<td>Not failure</td>
<td>32</td>
<td>24.7</td>
</tr>
<tr>
<td></td>
<td>Surface layer delamination</td>
<td>13</td>
<td>22.5</td>
</tr>
<tr>
<td></td>
<td>Surface layer cone</td>
<td>20</td>
<td>21.7</td>
</tr>
</tbody>
</table>
7 Summary and future step

The bond strength for wall and ceiling was same and floor application gave relatively low. But even the value for floor gives enough strength.

Difference of setting directions (upward, horizontal, downward) does not give big influence on bond strength. In jobsite tests at existing buildings, difference of concrete soundness seems to give bigger influence.

Uneven surface of structural members in existing buildings have impact on bond strength test. When confinement by shear plate is not enough, bond strength decreased down to about 90%.

Compared with laboratory test, bond strength of application to existing buildings amounted about 70–80% of potential value. Coefficient of variation of jobsite test is about twice that of laboratory test.

Based on jobsite test results, we plan to offer necessary subjects concerning operation quality control and jobsite control for injection type post-installed anchors.

Also for the public use of post-installed anchors, next problems still remain:

1. Study and review of operation quality control method of post-installed anchors
2. Jobsite control guideline of post-installed anchors

References:

1. AIJ Design Recommendation for composite construction
LONG-TERM CREEP BEHAVIOR OF RESIN-BASED INJECTION MORTAR SYSTEMS FOR ANCHORING APPLICATIONS

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ABSTRACT

For lifetime assessment of resin-based injection mortar systems in structural applications, a fundamental knowledge of the long-term deformation and failure behavior of such materials is required. This research work addresses the long-term deformation behavior by experimental determination of creep modulus curves for two commercially available epoxy resin (EP) and vinyl ester (VE) based injection mortar systems. The experimental approach involves an accelerated procedure for providing creep modulus data for these complex material systems, considering various material states of practical engineering relevance. Three reference material states were defined as upper and lower bound states, respectively, for the expected creep behavior. They were experimentally accomplished by appropriate temperature and moisture preconditioning. The upper bound reference state I represents and is referred to as the “(fully) cured & dry” state. Conversely, two lower bound reference states representing (a) reference state II “standard-cure – as received” and (b) reference state III “(fully) cured & wet” were defined. For reference state I (upper bound), a stress level of 10 MPa was chosen for the creep experiments at different temperatures, and creep modulus master curves for a temperature of 23°C and up to 50 years were deduced making use of the time-temperature equivalency principle. The creep modulus curves for the lower bound material reference states II and III were assessed based on modulus reduction factor deduced from short-term tensile experiments together with the functional creep characteristics of the upper bound reference state I. In this manner, a creep modulus function scatter band for the defined upper and lower bound conditions was achieved, which can be used for deformation modeling and simulation of EP and VE based injection mortar systems. While for the EP resin based injection mortar system the effect of curing state was found to be negligible (at least over the range investigated) with the effect of moisture content being quite evident, for the VE resin based injection mortar system, both material conditions profoundly affect the creep modulus properties.
1 Introduction

Polymer concretes have been used in the building industry since the mid-1920s and have established themselves as a material of choice for special areas and applications where the properties of conventional concretes are no longer sufficient. Particularly the cutting-edge construction industry is facing an increased amount of fixings in masonry. Compared to cement-based mortars they exhibit significant faster curing times, excellent surface adhesion, increased compression and tensile strengths as well as a better corrosion and chemical resistance. For the certification, simulation and the targeted use of these materials in the construction industry, a good understanding of the material behavior including knowledge on long-term properties is crucial.

In many applications, polymer based injection mortar systems are typically not fully cured. The mechanical properties and especially the creep moduli frequently show a strong influence on the degree of cure and especially also on moisture content. Under real ambient conditions, the direct construction of a creep master curve by time-temperature shifting is not possible due to the occurrence of many transient phenomena associated with post-cure and changing moisture uptake. A major objective of the present work was the development of creep modulus master curves in the linear viscoelastic regime for two resin based injection mortar systems as upper and lower bound reference curves accounting for changes in curing state caused by varying service temperatures and in moisture uptake caused by varying humidity. Therefore, a special test concept was designed and implemented.

2 General background

The use of polymeric materials in the building, construction and infrastructure sector represents the third largest plastics market. Applications range from piping, via insulation, sealing and gluing to improvement of concrete by adding polymeric resins and fillers. Of relevance to the present work, resin based systems are also used as injection mortar for the anchoring of dowels in bricks and concrete walls. Long-term mechanical properties and a reliable assessment of the service life are important aspects in the certification of available resin systems and in the development of improved novel resin systems. Hence, a comprehensive understanding of the constituent properties of the polymer resin matrix, the filler, and the resin/matrix interface, as well as any parameters influencing the composite material behavior is required. The relationship between process parameters, material structure and loading conditions of thermoset resins has been extensively studied over the last decades. For injection mortar systems based on epoxy resins and vinyl ester resins besides the temperature, the morphology, the curing degree and the moisture have the highest influence on the mechanical properties of these materials.

Epoxy and vinyl ester thermoset systems (i.e., in combination with particulate fillers and/or fibers) are being increasingly used for load-carrying applications. Hence, their long-term performance becomes a decisive design property. To obtain reliable predictions, information about the material behavior under long-term loading is required. For thermosets, the degree of curing and the moisture uptake are important parameters affecting the long-term behavior. Time-dependent material parameters determined by standardized creep tests are used for dimensioning mechanically loaded plastics in terms of their deformation behavior. To cover a wide time range (e.g., creep modulus data...
for loading times of up to 50 years), creep experiments to determine time-dependent creep moduli are frequently performed at various temperatures to subsequently generate creep modulus master curves by time-temperature shift techniques\textsuperscript{14,15}.

3 Experimental

The materials investigated included two commercially available epoxy resin (EP) and vinyl ester (VE) based injection mortars from different suppliers. Three reference material states were defined as upper and lower bound states, respectively, for the expected creep behavior. They were experimentally accomplished by appropriate temperature and moisture preconditioning. The upper bound reference state I represents and is referred to as the “(fully) cured & dry” state. It was achieved by curing the EP injection mortar system at 110°C for 24 hours, and the VE injection mortar system at 150°C for 96 hours. Conversely, two lower bound reference states were defined as follows: (a) reference state II “standard-cure – as received” (EP and VE: 23°C for 24 hours plus storage time until testing) and (b) reference state III “(fully) cured & wet” (EP: (fully) cured according to reference state I followed by 3 h immersion in water at 80°C; VE: (fully) cured according to reference state I followed by 30 h immersion in water at 80°C).

The experiments performed are including modulus measurements in tensile creep experiments and monotonic tensile experiments. Both experiments were performed using a universal testing machine with a maximum force of 20 kN in a temperature chamber (Zwick Roell Z020, Ulm/Germany). The creep experiments were conducted at six different temperatures ranging from room temperature up to 55°C. The clamping distance between the clamping grips was 115 mm and the gauge length for strain measurements was 50 mm. The applied creep stress was 10 MPa and the test duration was between 24 and 96 hours. Prior to testing, the specimens were kept for 60 min in the temperature chamber at the specific testing temperatures. The longitudinal and transversal strain was captured with extensometers and a camera system. An optical extensometer was used for strain determination. The evaluation of creep modulus was carried out on the digital image correlation (DIC) software Aramis professional. The creep modulus master curve of the injection mortar system was obtained by horizontal time-temperature shifting, commonly used for the creep evaluation of pipe materials\textsuperscript{16}. Due to material condition changes during long-term testing, for these material states, the measurement of reliable creep curves was not possible. The effects of incomplete curing and moisture were captured by reduction factors obtained via tensile tests. Tensile tests were performed at room temperature (23°C) with strain rates of $10^{-3}$, $10^{-4}$ and $10^{-5}$ 1/s. The clamping distance between the grips and the gauge length between the extensometers were identical to the creep tests. From the tensile tests, tensile secant modulus values were calculated between stress levels of 0.05 MPa and 10 MPa. The lower stress level of 0.05 MPa was selected to eliminate clamping effects and the upper value to enable a comparison of the tensile secant modulus with the creep modulus data. Based on the average results of these tensile tests, reduction factors were introduced to quantify the influence of incomplete curing and moisture. The reduction factor is obtained as the ratio of the average tensile secant modulus in the lower bound reference states II and III, respectively, to the average tensile secant modulus in reference state I. This definition allows for the assessment of the creep modulus function for the lower bound reference states at longer times by simply multiplying the creep modulus data of reference state I with the corresponding reduction factor.
4 Results and Discussion

Figure 1 depicts the creep test results for reference state I of EP and VE specimens. The single creep curves show a constant negative slope over time. Higher temperatures cause a vertical shifting of the creep modulus curves to lower values. While for EP at elevated testing temperatures of 50°C and 55°C the curves additionally show a higher slope, for VE similar slopes were obtained over the whole temperature range. The single creep modulus curves were shifted to a master curve at 23°C by using the time-temperature equivalency principle. Based on the experimental data of the tensile creep test temperatures between 23°C and 55°C, for EP a horizontal time-temperature shift up to 50 years is possible. The master curve at 23°C gives a creep modulus reduction of 68 % after 1 year and 82 % after 50 years. The shrinking of the VE specimens, which occurred during tensile creep testing and exposure to elevated temperatures (probably due to a combination of post-cure and/or moisture reduction), limited the shifted creep modulus master curve to times of one year which resulted in a creep modulus reduction of 42 %. The modulus after 50 years was predicted by linear extrapolation of the creep modulus master curve to be 53 %.

Figure 1: Logarithmic tensile creep curves for EP (left) and VE (right) at different temperatures in reference state “(fully) cured & dry” shifted to a reference master curve at 23°C.

The upper bound reference state I “(fully) cured & dry” is hardly reached during regular processing and service conditions. For the application of injection mortar systems, a quasi worst-case scenario such as covered by the lower bound reference states II and III caused by incomplete curing and moisture uptake, respectively, also has to be taken into account. In Figure 2, the tensile curves for the EP and VE injection mortar specimens tested in all three reference states at three different strain rates are shown. For both resins in reference state I “(fully) cured & dry” the highest tensile strengths with no significant influence of the applied strain rate were detected. Figure 3 shows the reduction factors, as defined in the experimental chapter. While for EP the lack of a tempering program (i.e., no exposure to elevated temperatures; reference state II “standard-cure – as received”) does not significantly influence the secant modulus (i.e., a reduction factor of 0.99), for VE a decrease of the secant modulus by 21 % resulting in a reduction factor of 0.79 was obtained. However, for both resins moisture uptake (reference state III) causes a decrease of the secant modulus resulting in a reduction factor of 0.70 and 0.83 for EP and VE, respectively.
Figure 2: Standardized stress-strain curves for three different strain rates tested in reference state I (top), reference state II (middle) and reference state III (bottom) for EP (left) and VE (right).

Figure 3: Reduction of average secant moduli between reference state I, reference state II, and reference state IIIb for EP (left) and VE (right).

In Figure 4, all EP and VE test results are combined into a creep modulus master curve. For the reference state I, the secant moduli obtained with tensile tests (filled red triangles) are combined with representative tensile creep test results (filled red squares) in one master curve. The data points were fitted with a second order polynomial fit. As mentioned above, for EP the construction of the master curve up to 50 years was possible by time-temperature shifting, while for VE the experimental time-temperature shift based master curve covered only a time up to 1 year. Hence, in the latter case the curve is extrapolated linearly or by curve-fitting, respectively, to 50 years. Also depicted is scatter band of the potential creep behavior (dashed area) accounting for the reference state corresponding to the most significant modulus reduction as detected by short-term tensile experiments (i.e., smallest reduction factor; reference state III “(fully) cured & wet” state for EP; reference state II “standard-cure – as received” for VE).
Figure 4: Creep modulus master curves for EP (left) and VE (right) incl. a scatter band for the potential creep behavior accounting for both lower bound reference states.

5 Summary and Conclusions

In the present paper, the long-term creep behavior of two commercial injection mortar systems, one epoxy (EP) based and the other vinyl ester (VE) based, is described. A novel test concept for constructing service-relevant creep modulus master curves for service-relevant material states (lower bound reference states II and III, not fully cured resin systems also covering post-cure and moisture uptake effects) was designed and implemented.

For the upper bound reference state I “(fully) cured & dry” the initial value of the short-term modulus data (1 s) for the EP injection mortar system decreased by 68 % and 82 % after 1 year and 50 years, respectively. The lower bound reduction factors with which the upper bound data have to be multiplied were found to range from 0.7 (reference state III) to 0.99 (reference state II). Correspondingly, for the upper bound reference state I “(fully) cured & dry” the initial value of the short-term modulus data (1 s) for the VE mortar system decreased by 42 % after 1 year and by 53 % after 50 years. The lower bound reduction factors with which the upper bound data have to be multiplied were found to range from 0.79 (reference state II) to 0.83 (reference state III).

6 Acknowledgment

This research work was performed at the Johannes Kepler University Linz – Institute of Polymeric Materials and Testing (IPMT) in cooperation with the Christian Doppler Laboratory: Life Cycle Robustness in Fastening Technology (LiCRoFast). The project was funded by the companies fischerwerke GmbH & Co. KG and Hilti AG and by the Federal Ministry of Science, Research and Economy (BMWFW) and the Christian Doppler Research Association.

References:


CONCRETE CREEP EFFECT ON BOND STRESS IN ADHESIVE FASTENING SYSTEMS

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ABSTRACT

Due to serious accidents caused by anchor failures, it became obvious that a more fundamental insight into the long-term performance of fastening systems under sustained load is needed, even though the failures were ultimately attributed to human error. Current design and approval guidelines for adhesive anchors are based on an incomplete understanding of the deformation behavior under sustained loads, compensated by conservative assumptions. Thus, a more precise forecast of the long-term performance of adhesive fastening systems under sustained load is required to ensure their safe but also efficient design. Anchor systems are comprised of different materials, namely concrete, steel, and polymer-based mortars. Therefore, the contribution of each component on the entire system performance has to be investigated and quantified. The present study aims at quantifying numerically the concrete creep effect on the effective bond stress distribution. A model that couples hydration, diffusion, and heat transport with the concrete creep response is used. Once the calibration of the model is completed and independently validated based on concrete only tests, a case study on bonded anchor systems is performed. In particular, the time evolution of bond stress of bonded anchors, for two different loading configurations, confined and unconfined, are investigated. Preliminary results show a progressive divergence from the initial bond stress condition, in both configurations.

1 Introduction

Fastening systems are widely used in structural engineering. Typical applications concerning the primary structure range from the connection of precast elements and different structural members to the connection of new structural elements to existing structures. Fastenings are further characterized by a large ratio between cost of the individual fastener and total economic loss in case of failure. Therefore, the necessity for models emerges that are capable of predicting, with quite high accuracy, the system behavior in time horizons of many decades – times that exceed by far the practical experience with these systems as well as the experimental capabilities of laboratories.

For an accurate forecast of the long-term behavior of fastening systems, a thorough understanding of deteriorating mechanisms of all involved materials is necessary. Anchors can be categorized in two main groups regarding their load transfer mechanism: mechanical and chemically bonded systems.
The mechanical anchors transfer loads either by mechanical interlock or by friction between steel and concrete. In bonded anchor systems adhesion is achieved using a bonding agent, e.g. organic or inorganic mortar, and the load is transferred by means of bond\(^1\). In this contribution, the focus lies on chemical anchors even though some aspects apply also to the former group. For bonded anchors, the long-term behavior is determined by the visco-elastic behavior of both mortar and concrete as well as the potentially progressive development of damage leading to stress redistributions. Understanding and characterizing the contributions to the long-term response, individually as well as in interaction, represents the first step in understanding the system. In particular, the concrete creep effect on bonded anchor systems and especially on the bond stress distribution in course of time is studied in this contribution.

According to Cook\(^2\), a uniform bond law model can be assumed for the design of a single bonded anchor system. For a ratio of embedment depth, \(h_{\text{eff}}\), to anchor diameter, \(d\), between 4 and 20, and 50 mm maximum value of the latter, a bond stress strength, \(\tau\), is related to the pull-out strength \(N\) as

\[
N = \tau \pi d h_{\text{eff}}
\]

This equation introduces a maximum bond strength\(^3\) which can be utilized as

\[
\tau = 4.3 f_{\text{cc}}^{0.5} h_{\text{eff}}^{0.5} / d
\]

where \(f_{\text{cc}}\) the concrete cube characteristic strength. When an anchor system is loaded under sustained load stress redistribution is likely expected, due to viscoelasticity and potential damage. Therefore, the investigation of bond stress under sustained loading has a high significance for a safe design.

A bonded anchor system is comprised by different components – mortar, concrete – that exhibit viscoelastic response even at room temperatures. So far, a wide used method concerning the viscoelasticity of such a system is the so-called Findley approach\(^4\). This method fits a power law to data of a system creep test and then extrapolates in time. A more refined approach requires the separation of the visco-elastic system behavior into the components’ contributions. Following this, concrete and mortar creep can be characterized individually and then “assembled” to a system response.

In this paper, a numerical study of the concrete creep effect on the bond stress distribution of a bonded anchor system is carried out. For concrete creep the Microprestress Solidification theory\(^5\) (MPS) is used coupled with the Hygro-Thermo-Chemical model\(^5\) (HTC), in a discrete concrete framework. The mechanical response is represented by the Lattice Discrete Particle Model\(^6\) (LDPM). Calibration and validation of the aforementioned models is accomplished using different test data. In particular, standard cubes, cylinders and prisms are tested to determine compressive strength \(f_c\), Young’s modulus \(E_c\), Brazilian splitting tensile strength \(f_t\), and fracture energy by three point bending tests. Concrete creep and shrinkage are calibrated based on standard compressive creep and shrinkage tests. Additionally, anchor pull out and sustained load tests are performed, for the quantification of a bond – slip law and an implicit calibration of a viscoelastic bond law to describe mortar creep.
2 Numerical framework

The different chemical and physical processes that occur in concrete in course of time impose the necessity of a combined multi-scale multi-physics approach in combination with a model capable of describing the inherent heterogeneity of the materials.

2.1 Hygro-thermo-chemical model

In this investigation, the hygro-thermochemical model (HTC), is used, which formulates and solves the problems of moisture transport, hydration and temperature evolution. The complicated task of combining all the aforementioned processes in a formulation is carried out by selecting as state variables just relative humidity and temperature. Their time evolution is described by the following partial differential equations

\[
\nabla \cdot (D_h \nabla h) - \frac{\partial w_e}{\partial h} \frac{\partial h}{\partial t} \dot{a}_c - \frac{\partial w_e}{\partial a_s} \dot{a}_s - \dot{w}_n = 0
\]

(3)

\[
\nabla \cdot (\lambda_t \nabla T) - \rho c_t \frac{\partial T}{\partial t} + \dot{a}_c c_{Q_c} + \dot{a}_s s_{Q_s} = 0
\]

(4)

with \( D_h \) = permeability, \( h \) = relative humidity, \( w_e \) = evaporable water, \( a_c \) and \( a_s \) cement hydration and silica fume reaction degrees, \( w_n \) = non-evaporable water, \( \lambda_t \) = heat conduction coefficient, \( T \) = temperature, \( \rho \) = concrete density, \( c_t \) = isobaric heat capacity, \( c \) and \( s \) cement and silica fume content, \( Q_c \) and \( Q_s \) cement hydration and silica fume reaction enthalpies, respectively.

2.2 Microprestress Solidification theory

Concrete creep is modelled according to the MPS formulation. According to it and using the principle of superposition, a Kelvin chain, which describes the viscoelasticity of the material, is used in series with a spring, and an aging dashpot, that accounts mostly for the long-term behavior. Additionally, shrinkage and thermal strains can be imposed. The rheological model is shown in Figure 1.

![Rheological model of MPS](image)

Following the logic of the above rheological model the total strain could be expressed as the addition of all strain components, thus

\[
\varepsilon_{tot} = \varepsilon_* + \varepsilon_v + \varepsilon_f + \varepsilon_{sh} + \varepsilon_T
\]

(5)
where \( \varepsilon_{\text{tot}} \) total strain, \( \varepsilon^{*} \) the instantaneous response, \( \varepsilon_{v} \) the viscoelastic strain, \( \varepsilon_{f} \) the pure viscous flow, \( \varepsilon_{sh} \) the shrinkage strain, \( \varepsilon_{T} \) the thermal strain. The instantaneous and visco-elastic strains are given by Equations 5-6, respectively

\[
\varepsilon^{*} = \frac{\sigma}{E_0} \tag{6}
\]

\[
\varepsilon_{v} = \frac{\gamma}{V(a_c)} \tag{7}
\]

where \( \gamma \) is the non-aging creep strain, and \( V(a_c) \) is the volume fraction of present cement gel. Function \( V(a_c) \) is necessary to introduce the aging viscoelasticity, since this is not included in the chain. The volume of reacted hydrate can be approximated as a power law in terms of cement hydration degree, i.e. the amount of cement that has reacted over the available cement content. For this concrete an exponent of 1.4 is used, that is in very good agreement with short time creep data of different loading ages. The non-aging creep strain \( \gamma \) can be approximated by a Dirichlet series, equation (7), assuming a logarithmic microcompliance function for \( t' \) age of loading, equation (8).

\[
\gamma(t) = \sigma \sum_{\mu=1}^{N} \frac{1}{E_\mu} \left( 1 - e^{-\left(t-t'\right)/\tau_\mu} \right) \tag{8}
\]

\[
\Phi(t-t')=q_2 \ln[1+(t-t')^{0.1}] \tag{9}
\]

with \( E_\mu \) and \( \eta_\mu \), the moduli and viscosities of springs and viscosities of unit \( \mu \) of the chain, \( \tau_\mu = \eta_\mu / E_\mu \) the characteristic time, \( q_2 \) a material parameter that has to be calibrated, and \( N \) the number of the chain units, which for this study is equal to 10. Finally, the viscous flow can be expressed as

\[
\varepsilon_{f} = q_4 \frac{\sigma}{\eta_f} \tag{10}
\]

Where \( \eta_f = 1/cS \) the viscosity of the aging dashpot, with \( c \) a constant, \( q_4 \) a parameter to be calibrated, and \( S \) the microprestress which can be calibrated using drying creep data.

### 2.3 Lattice discrete particle model

The aforementioned creep model is implemented in the framework of LDPM, which is a mesoscale discrete concrete model. The formulation follows the assumption of spherical aggregates, the particles. Around them a lattice system is created by the line segments that are connecting the particle centers. A system of polyhedral cells, which interact through triangular facets, represents the mesoscale concrete geometry. For more details on the constitutive equations one can refer to Cusatis et al.\(^6\).

### 3 Concrete tests – calibration and validation

In order to calibrate the models, various concrete tests for two different concrete batches are performed. The mix design, and cube compressive strength, \( f_{c28} \), is shown in Table 1.
### Table 1: Mix design and material properties of tested concretes

<table>
<thead>
<tr>
<th>Batch</th>
<th>A</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement content [kg/m³]</td>
<td>240</td>
<td>280</td>
</tr>
<tr>
<td>water/cement</td>
<td>0.81</td>
<td>0.64</td>
</tr>
<tr>
<td>Aggregate/cement</td>
<td>8.72</td>
<td>7.44</td>
</tr>
<tr>
<td>$f_{c,28}$ [MPa]</td>
<td>25.99</td>
<td>32.98</td>
</tr>
</tbody>
</table>

In detail, a full material characterization using different tests are performed, including cube and cylinder compression, three point bending and Brazilian splitting at different ages. In Figure 2 the calibration of material properties for cube compressive and three point bending fracture tests is shown.

![Figure 2: Calibration of material properties of concrete for batch A tested at 28 days for (a) cube compression and (b) three point bending.](image)

Additionally, creep and shrinkage tests are performed at different ages for both drying and sealed specimens. Cylindrical specimens of 150mm diameter and a length of 300mm are kept in saturated lime water bath until the age of testing. Basic creep and autogenous specimens are sealed with aluminum foil. Creep specimens are loaded at 30% of strength at that age, using the creep tower that is shown in Figure 3. Creep and shrinkage strains are measured using strain gauges of 50mm length, installed parallel to the cylinder axis on the mantle in a 120° degree configuration, as it is shown in Figure 3. The experimental results, as well as the numerical curves are shown in Figure 4.
Additionally, for batch B, sustained load tests on bonded anchor systems are performed for two configurations, a confined and an unconfined, as it is shown in Figure 5. Bonded anchors of 16 mm radius (M16), were installed in cylindrical specimens of 250 mm length and diameter of 300 mm and 450 mm, respectively. The embedment depth was 100 mm. For the unconfined configuration, a metallic ring of 300 mm inner diameter and for the confined set up a confinement plate were used. Both systems were loaded to and kept at 30% of the anchor load capacity using a system of hydraulic jacks. The load was kept at the desired values within a scatter of ±2%. In Table 2 the values of pull-out maximum load and the sustained load levels for both configurations are presented.

Table 2: Pull-Out capacity of anchor system tested at 66 days

<table>
<thead>
<tr>
<th>Set up</th>
<th>Confined</th>
<th>Unconfined</th>
</tr>
</thead>
<tbody>
<tr>
<td>$N$ (kN)</td>
<td>136.6</td>
<td>89.6</td>
</tr>
<tr>
<td>Sustained load</td>
<td>41.0</td>
<td>26.9</td>
</tr>
</tbody>
</table>
Figure 4: Calibration of creep model on age of 3 days (a) and validation on age of 28 days (b), calibration of shrinkage coefficient on drying, exposed at 3 days, and sealed specimens (c) and validation on drying specimen exposed at environmental conditions at 28days (d).

Figure 5: Sustained load tests of bonded anchor for an unconfined (a) and a confined configuration (b), respectively.
4 Anchor models

The numerical study was carried out using the geometries that are shown in Figure 6. In both cases the embedment depth was 100mm, while the model geometries were the same as the experimental of Figure 5. Concrete is simulated with LDPM, while the anchor rod is taken as beam element. Additionally, the adhesive material is simulated as an interface layer, assuming a multilinear bond-slip law.

![Figure 6: Model geometries for (a) unconfined and (b) confined configuration](image)

4.1 Bond law

The connection between steel and concrete, i.e. the mortar response is simulated through a bond-slip law. For the calibration of the bond law parameters, pull-out tests in a confined configuration are used. Furthermore, unconfined pull-out test data is used to validate the calibration of the bond law. For more details one can refer to a companion paper. The visco-elastic nature of the mortar is not accounted for at this time.

![Figure 7: Load transfer mechanism in bonded anchor system (a) and schematic multilinear stress-slip law taken for concrete steel interface (b)](image)
4.2 Concrete creep

Having calibrated the mechanical and creep model of concrete as well as the interface parameters, a study on the effect on the bond stresses is carried out. Firstly, a pure predictive study for concrete A is performed. Both configurations are simulated using a sustained load of 40% of pull-out capacity. Simulations of sustained loading up to 50 years, for the confined configuration, show a stress redistribution of bond stresses and a trend of divergence from the uniform bond stress law.

Figure 8: Simulated normalized bond stress $\tau$ for unconfined (a) and (b) confined configuration for different time instants

In case of the unconfined configuration the divergence becomes more rapid, after 12 years already it tends to unload at the top of the bar. Furthermore at the bottom of the anchor the stress redistributes gaining an additional value of around 40%.

4.3 Viscoelastic interface

In order to simulate more precisely the creep response of an anchor system, also the viscoelastic response of the mortar layer, i.e. the interface in this discretization has to be simulated. Particularly, a Kelvin chain formulation of five units is used imposing the creep strain in the slip response of the interface. An inverse calibration of mortar viscoelasticity has been done. Parameters of the viscoelastic interface are calibrated using experimental data of sustained load tests for batch B, considering the already calibrated creep contribution of concrete. Since the experimental data are limited to a time span of three months, insights into the long-term response are limited. A more accurate method could be the characterization of the mortar viscoelasticity based on suitable tests the pure mortar and then the transition to a viscoelastic interface. Nevertheless, it is interesting to investigate the different effect of this viscoelastic interface on the bond stress distribution. In Figure 9 is shown the inverse calibration of the interface viscoelastic model using experimental data of sustained load tests. Also is shown the different effect of concrete creep and mortar layer creep. The numerical study shows that concrete creep has an antagonistic effect to that of a viscoelastic mortar layer. Particularly the concrete creep redistributes the stresses along the anchor rod in a diverging
manner, in course of time. On the other side, in the case where only the interface viscoelasticity is taken into account, the simulations show a trend of more uniformly distributed stresses. In the case where both material contribute in the viscoelastic response of the system, a combined effect is obtained numerically. In early age the contribution of mortar layer appears to be the dominant, while at later ages, a slight divergence is observed which can be explained by the large characteristic time of concrete creep. The mortar viscoelasticity was calibrated using an inverse system approach, and only short term data, fact that limits the conclusions. However a viscoelastic interface based on mortar creep tests in combination to concrete creep model could describe better the system and predict more accurate its response.

Figure 9: Sustained load test data and numerical creep response regarding only concrete creep and concrete plus interface viscoelasticity (a), and numerical bond stress profiles for 3 different cases, (b) for concrete creep, (c) for interface creep, and (d) for both mechanisms loaded at 30% of pull out capacity.
5 Conclusion

In this contribution, the effect of concrete creep on the long-term bond stress distribution of bonded anchor systems is studied numerically. The concrete creep appears to lead to a divergence from a uniform bond stress, even for the confined case. This divergence is enlarged in course of time. When also the viscoelasticity of mortar is taken into account by means of a viscoelastic interface, the bond stress between steel and concrete tends to initially approach a more uniform bond stress distribution, while afterwards shows a slight divergence. Obviously, the two creep contributions of concrete and mortar have competing effects on the bond stress distribution. For more definitive statements additional experimental data and numerical simulations are required.

6 Acknowledgement

The financial support by the Austrian Federal Ministry of Economy, Family and Youth and the National Foundation for Research, Technology and Development is gratefully acknowledged, as is the additional support by our industrial partners.

References:

The use of bonded anchors in engineering practice is continuously increasing. However, due to the complexity of the load-transfer mechanism and use of new chemical adhesives, their safety under sustained load is still not clear and it is the topic of intensive research. In the present paper the results of a 3D finite element parametric study for a single bonded anchor loaded under sustained tensile load are presented and discussed. The numerical model is first calibrated based on the experimental tests and subsequently the parametric study is carried out. The influence of geometry, fracture properties and creep of concrete and chemical adhesive (epoxy) on the behavior of anchors under sustained load is investigated. The results of the study show that fracture of concrete and its interaction with creep of concrete has dominant influence on the resistance and behavior of anchors under sustained load. Moreover, the behavior of bonded anchors under sustained load is more sensitive if the ratio between the strength of chemical adhesive and concrete increases. It is shown that the most critical situation is the use of high quality polymer in a low quality concrete. With this respect further experimental and numerical studies are needed.

1 Introduction

Bonded anchors transfer the load from metallic anchor or rod into a concrete member over the chemical adhesive (e.g. epoxy). Their behavior is dependent on embedment depth, concrete properties, properties of steel used for the threaded rods, properties of chemical adhesive (strength and stiffness) and boundary conditions. Similar to mechanical anchors, design of bonded anchors is based on the Concrete Cone capacity method. However, with respect to the sustained load there are two important differences between mechanical and bonded anchors: (i) Time deformation of mechanical anchors (headed stud, post-installed undercut or expansion anchors) is due only to the contribution of concrete. In bonded anchors, instead, time deformation comes from the contribution of concrete and chemical adhesive. Therefore, it is not surprising that the behavior of bonded anchors under sustain load is more sensitive than the behavior of mechanical anchors. (ii) Mechanical anchors transfer the load from the anchor into the concrete as a point load relatively deep into a concrete member. However, bonded anchors transfer the load over the chemical adhesion through the entire anchor length. From the fracture mechanics point of view, failure mechanism of mechanical anchors belongs into the category of the so called negative geometries, i.e. with the increase of the crack length, stress intensity factor at the crack tip decreases. Therefore, when at
constant load the crack length increases, for instance due to creep of concrete, resistance is not decreasing. However, in bonded anchors there is a redistribution of stresses between concrete and chemical adhesive over the entire embedment depth, which generally leads to decrease of anchor resistance under sustained load.

Due to the above mentioned differences between mechanical and bonded anchors, the design of bonded anchors with respect to the sustained load is more demanding than the design of mechanical anchors. It is still not clear how the effect of creep of concrete and chemical adhesive should be accounted for in order to design save and economical fastenings. The main aim of the present study is to investigate, in the qualitative sense, the factors that are responsible for the failure of bonded anchors under sustained load.

2 Experimental tests

The behavior of epoxy bonded anchors under sustained load at ambient temperature of 23°C was recently experimentally investigated by Chiurazzi. The results of experimental tests are used to calibrate the numerical model. The geometry of the tested specimen is shown in Fig. 1.

![Geometry of the tested specimen](image)

Table 1 Summary of the basic material properties

<table>
<thead>
<tr>
<th>Mechanical properties</th>
<th>Concrete</th>
<th>Polymer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity [GPa]</td>
<td>30.0</td>
<td>3.5</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.18</td>
<td>0.35</td>
</tr>
<tr>
<td>Uniaxial Compressive strength [MPa]</td>
<td>26.0</td>
<td>60</td>
</tr>
<tr>
<td>Uniaxial tensile strength [MPa]</td>
<td>2.6</td>
<td>60</td>
</tr>
<tr>
<td>Fracture energy [J/m²]</td>
<td>80.0</td>
<td>/</td>
</tr>
<tr>
<td>Limit strain</td>
<td>/</td>
<td>0.50</td>
</tr>
<tr>
<td>Creep factor (linear creep)¹</td>
<td>2.0</td>
<td>2.0</td>
</tr>
</tbody>
</table>

¹ Assumed values of creep factors

Both, the short term tests and sustained load tests, have been experimentally performed. The sustained load was applied for three different load levels: 85%, 75% and 65% of the peak load obtained from the short term loading tests. The material properties from the experimental test are
summarized in Tab. 1. The properties listed in Tab. 1 are also used in the numerical analysis. In the analysis the steel is taken as linear elastic with Young’s modulus of 200 GPa and Poisson’s ratio 0.33. Note, however, that the properties for polymer and linear creep factor for concrete and polymer, specified in Tab. 1, are obtained through the calibration of the numerical model, as discussed below.

3 Numerical studies

3.1 Constitutive law and FE discretization

Numerical simulation is performed using 3D finite element (FE) code MASA. The constitutive law for concrete is the microplane model with relaxed kinematic constrain. Polymer (epoxy) is modeled by the recently proposed microplane model for metallic kind of materials, assuming to be friction insensitive. Principally, creep of concrete and polymer is simulated based on the linear rate-type creep law (generalized Maxwell chain model) with eight age dependent units. It is coupled into a series with the microplane model (Fig. 2). In this way the total strain is decomposed into elastic, damage (microplane) and creep component.

![Diagram of microplane and generalized Maxwell chain model](image)

**Fig. 2** Serial coupling of microplane model (left) and generalized Maxwell chain model (right)

Creep strain from the Maxwell chain model is assumed to be linear proportional to stress. For concrete and polymer this is approximately valid if the stress level compared to strength is relatively low. However, at higher stress levels there is the interaction between non-elastic strains (creep) and damage, which leads to progressive creep deformations, i.e. non-linear creep (creep-fracture interaction). To account for non-linear creep, here is proposed relatively simple approach. The linear creep strain obtained from the generalized Maxwell chain model is multiplied by the function $f(\sigma)$ that depends on the relative stress level and accounts for the damage induced creep:

$$\varepsilon_{\text{creep}} = f(\sigma) \varepsilon_{\text{creep,linear}}$$  \hspace{1cm} (1a)

$$f(\sigma) = 1.0 \hspace{1cm} \text{for} \hspace{0.5cm} \sigma_{\text{vM}} \leq \alpha f_{c,R}$$  \hspace{1cm} (1b)

$$f(\sigma) = 1.0 + 50 \left( \frac{(\sigma_{\text{vM}} - \alpha f_{c,R})}{(1-\alpha f_{c,R})} \right)^2 \hspace{1cm} \text{for} \hspace{0.5cm} \alpha f_{c,R} < \sigma_{\text{vM}} \leq f_{c,R}$$  \hspace{1cm} (1c)

with

$$f_{c,R} = f_c (0.80 - (3 \sigma_V / 5 f_c))$$  \hspace{1cm} (1d)
where $\sigma_{M} = \text{von Mises stress}$, $\sigma_V = \text{volumetric stress}$, $f_c = \text{uniaxial compressive strength}$, $f_{c,R} = \text{uniaxial confined compressive strength}$ and $\alpha = \text{proportionality limit}$.

![Fig. 3 3D FE discretization $h_{ef}/d = 6$: (a) confined specimen and (b) unconfined specimen.](image)

The typical FE discretization of the experimentally tested geometry is shown in Fig. 3a. The discretization is performed using 3D eight (polymer, anchor and external steel cylinder) and four nodes finite elements (concrete and supporting steel plate). Figure 3b shows the model in which the internal steel plate is removed and supports are imposed at the top of the external steel cylinder (unconfined specimen). The load is applied at the top of the anchor, either through displacement control (instantaneous loading) or by load control (sustained load). To assure results independent of the element size the regularization based on the simple crack band method is employed.

### 3.2 Calibration of the model

In the first step the numerical model is calibrated based on the test results (confined specimen). The mechanical properties of concrete were known (see Tab. 1). However, the properties of polymer (epoxy) are calibrated by fitting the test data. The typical experimental load-displacement curves for instantaneous load are shown in Fig. 4a and compared with the numerically predicted curve. In the experiment the failure is due to the brittle failure of polymer. The relatively brittle failure takes place short after the peak load (approximately 97 kN) and it takes place after the strain limit is reached. By fitting the test results it turns out that the best fit is obtained for uniaxial strength of polymer of 60 MPa with limit strain of 0.50 (see Tab. 1). The corresponding stress-strain curve for uniaxial compression and tension is plotted in Fig. 4b.

To calibrate the model for the sustained load, the experimental results for confined specimen loaded with two different load levels, 85% and 65% of ultimate short term resistance, are simulated. Based on the fitting of the test results the reasonably good agreement is obtained for linear creep factor of
Maxwell-chain model $\phi = 2$ and non-proportionality limit $\alpha = 0.70$, for both concrete and polymer (Fig. 6).

Fig. 4 Short term loading: (a) Load-displacement curves and (b) Uniaxial stress-strain curve for polymer

Fig. 6 Numerical vs. experimental results: (a) 85% of short term resistance and (b) 65% of short term resistance

3.3 Parametric studies

After the calibration of the numerical model a parametric study are performed. In the first part of the study the material properties for concrete and polymer are kept constant (see Tab. 1) and only embedment depth is varied: $h_e/d = 4, 6$ and 8. The analysis is performed for confined and unconfined conditions and the level of sustained load is varied from 30% to 85% of the short term resistance.

The typical load-displacement curves for short term loading are shown in Fig. 7. By reducing the sustained load level from 85% to 30%, the time to failure gradually increases and under a certain load level (strength under sustained load) no failure occurs. Similar results are obtained for all configurations and for both confined and unconfined specimens. However, the limit on the sustain strength is higher for unconfined specimen, i.e. 50% for confined and 65% for unconfined specimen (see Fig. 8). The reason is due to the fact that for confined specimen the load transfer from the
anchor into concrete is localized along the anchor length and von Mises stresses are approximately constant. Consequently, no redistribution of shear stresses over the anchor length is possible. In unconfined specimen, instead, the distribution of von Mises stresses over the anchor length is not uniform (see Fig. 9b). Therefore, due to the creep of concrete and polymer, the redistribution of stresses is possible with the consequence that the strength under sustained load is higher, i.e. 65% of short term loading resistance instead of 50% in case of confined anchor. The influence on the sustained strength in unconfined specimen is stronger especially for higher load levels. The reason is due to the fact that for smaller embedment depth the possibility for redistribution of stresses due to creep-fracture interaction is lower than in case when the embedment depth is larger.

![Fig. 7 Load-displacement curves for different embedment depth: (a) confined and (b) unconfined specimen](image)

The typical distribution of damage in concrete in terms of maximum principal strains, which corresponds to the crack width of approximately 0.25 mm, is for sustained load level of 65% and $h_{ef}/d = 8$ shown in Fig. 10 for confined and unconfined specimens. Because of the non-elastic deformations due to creep of concrete and polymer there is an increase of concrete damage in time, which finally leads to failure. This is the case for both, confined and unconfined specimens. However, in the confined specimen damage is localized in the concrete elements along the
embedment depth with the stresses level that is almost constant over the entire length (see Fig. 9a). The failure of confined anchor is due to the pull-out. In unconfined specimen maximum damage is observed at the bottom of the anchor. Due to creep it propagates in time along the anchor depth and inclined cracks (concrete cone) forms with the final failure mode that is a combination of the pull-out and concrete-cone failure. Consequently, in contrary to the confined anchor, for unconfined anchor the redistribution of stresses along the anchor depth is possible, which explains why the sustain strength is higher. As can be seen from Fig. 9b, due to the redistribution of stresses, the peak stress is moving from the bottom of the anchor into direction of concrete surface. The maximum von Mises stress in polymer in both cases is approximately the same, however, for unconfined anchor it is located only at the bottom of the anchor whereas for confined anchor maximum stress is distributed over the entire anchor length.

Fig. 9 Distribution of von Mises stresses in the polymer over the embedment depth for $h_{ef}/d = 6$ and sustained load level 85%: (a) confined and (b) unconfined

<table>
<thead>
<tr>
<th>Case</th>
<th>Concrete</th>
<th></th>
<th>Polymer</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Uniaxial compressive strength (MPa)</td>
<td>Tensile strength (MPa)</td>
<td>Compressive &amp; tensile strength (MPa)</td>
</tr>
<tr>
<td>Case 0 (Experimental test)</td>
<td>26.0</td>
<td>2.60</td>
<td>60.0</td>
</tr>
<tr>
<td>Case 1</td>
<td>50.0</td>
<td>5.00</td>
<td>60.0</td>
</tr>
<tr>
<td>Case 2</td>
<td>50.0</td>
<td>5.00</td>
<td>30.0</td>
</tr>
<tr>
<td>Case 3</td>
<td>26.0</td>
<td>2.60</td>
<td>30.0</td>
</tr>
</tbody>
</table>

From the above numerical results, it is obvious that creep of concrete and polymer significantly influence the sustained resistance of chemical anchors. However, it is not clear whether creep of concrete, polymer or both is responsible for relatively strong degradation of resistance under sustained load. To investigate this, the parametric study for the case $h_{ef}/d = 6$ and sustained load of 85% and 65% is carried out. In the study the following cases are investigated: (1) creep of both, concrete and polymer, is considered; (2) only creep of polymer; (3) only creep of concrete and (4) creep of both materials but concrete is assumed to be linear elastic. The results are plotted in Fig. 11
in terms of displacement history curves. They clearly indicate that in most cases the interaction between creep of concrete and non-linear behavior of concrete (damage and cracking) is responsible for strong reduction of the strength under sustained load. Only in case of relatively high sustained load (85%) and unconfined anchor the influence of creep of concrete and polymer is practically the same and almost independent of damage in concrete.

<table>
<thead>
<tr>
<th>Confined specimen</th>
<th>0.24 h</th>
<th>1.56 h</th>
<th>269 h</th>
<th>2668 h</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unconfined specimen</td>
<td>0.24 h</td>
<td>1.56 h</td>
<td>269 h</td>
<td>266668 h</td>
</tr>
</tbody>
</table>

Fig. 10 Development of damage with time ($h_{ef}/d = 8$, 65% of sustain load), confined (top) and unconfined (bottom) (red zone corresponds to crack width of 0.25 or greater)

In further parametric study the influence of material properties (uniaxial compressive and tensile strength) on the resistance of anchors under sustained load is investigated. The material properties for the analyzed cases are summarized in Tab. 2. Note that all other properties are the same as specified in Tab. 1. The analysis is carried out for $h_{ef}/d = 6$ (confined and unconfined anchors).

The short term load-displacement curves for all cases, as summarized in Tab. 2, are plotted in Fig. 12. As expected, with increasing strength of concrete and polymer the anchor resistance increases. For the confined specimen the resistance is directly proportional to the strength of polymer and concrete has almost no influence.
The predicted displacement time history curves are plotted in Fig. 13. The results clearly show that the ratio between the strength of polymer and concrete significantly influence the response of anchor under sustained load. If this ratio is high (Case 0) the resistance of anchor under sustained load is
significantly reduced. However, if it is low (Cases 2 and 3) the anchor strength is not much sensitive to the creep induced reduction of resistance. Furthermore, the stress level also strongly influences the sustained strength. Principally with increase of the stress level, at approximately the same ratio between the strength of polymer and concrete, the sustained resistance decreases (compare Case 1 and Case 3 in Fig. 12).
4 Conclusion

In the present study the influence of creep of concrete and polymer of chemical anchors is numerically investigated. Based on the results of numerical study the following can be concluded. (1) The employed numerical model, which is calibrated based on the experimental tests, is able to realistically replicate the response of anchors for instantaneous and sustained load; (2) It is demonstrated that the behavior and resistance of anchors under sustained load is mainly controlled by creep of concrete that interacts with damage; (3) For confined anchors no redistribution of shear stresses over the anchor length is possible. However, in case of unconfined anchors such redistribution is possible with the consequence that for relatively high load level the time to failure increases. Moreover, the sustained strength for unconfined anchors is higher; (4) For unconfined anchors time to failure decreases with decrease of embedment depth. Confined anchors are less sensitive on the embedment depth due to the fact that the stress distribution over the anchor length is almost constant; (5) The most critical situation for bonded anchors occurs when the strength of polymer is much higher than that of concrete. In such cases stresses and damage of concrete is relatively high, which cause significant creep induced damage of concrete (non-linear creep) and leads to strong reduction of resistance; (6) Further experimental and numerical studies are needed in order to investigate the influence of material properties, geometry and environmental conditions on the resistance of bonded anchors under sustained load.

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STUDIES ON CREEP DEFORMATION OF INORGANIC-TYPE POST-INSTALLED BONDED ANCHOR

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ABSTRACT

Organic-type post-installed bonded anchors, such as epoxy resin and epoxy acrylate resin, are mostly used worldwide even if inorganic-type post-installed bonded anchors have an advantage of durability, including heat and alkali resistance. Little has been reported on the durability performance of inorganic-type post-installed bonded anchors under sustained load, and many unknown points in their characteristics exist. In 2010, the injection type for an inorganic-type post-installed bonded anchor was developed in Japan. The present study involved the conduction of creep tests on organic-and inorganic-type post-bonded anchors and estimated the creep deformation for long term using exponential and logarithmic functions from the measured deformation. As regards the creep deformation of the inorganic-type post-installed anchor, the estimated displacement of the free end at 50 years was larger than that at the ultimate load. However, that of the loading end at 50 years was smaller. The evaluation at the free and loading ends greatly differed because the displacement of the inorganic-type anchor at the bond strength test was remarkably smaller than that of the organic type. Hence, a slight difference of the displacement greatly affected the estimated creep deformation. Furthermore, the estimated creep deformation by the exponential function at the age of 50 years was approximately 1.3 times larger than that by the logarithmic function. With regard to the estimated equation, the application for the inorganic-type anchor was assumed to be appropriate for the logarithmic function and that for the organic-type was appropriate for the exponential function.

1 Introduction

The ceiling of the Ted Williams Tunnel in South Boston, Massachusetts, USA collapsed on July 10, 2006. The National Transportation Safety Board subsequently released a report about the accident and mainly attributed the collapse to “epoxy creep”. The concrete ceiling panels inside the Sasago Tunnel in Japan also collapsed on December 2, 2012. These incidents call for an investigation as regards the clear durability of post-installed anchors.
In the past years, Japan has used more than 500 million post-installed anchors every year. The injection-type, inorganic-type post-installed bonded anchor was developed in 2010 from the viewpoint of durability. Inorganic types have a short history, thereby leading to many unknown points in their characteristics. In addition, little has been reported on the durability performance of inorganic-type post-installed bonded anchors under a sustained load.

Reports on the creep behaviour of post-installed anchors have recently increased in gradual rate. Yano et al. studied the creep behaviour of the post-installed anchors of polyester resin, whilst Nakano et al. investigated the anchors of epoxy ester. Ando et al. reported on the creep method of post-installed anchors. The creep behaviour of the organic-type bonded anchors has been summarised in the book entitled “Anchorage in Concrete Construction”.

Nevertheless, few studies focused on the creep behaviour of inorganic-type post-installed bonded anchors. The present study conducted sustained load tests on organic- and inorganic-type post-bonded anchors, and estimated the creep deformation using exponential and logarithmic functions from the measured deformation. Their applicability was also examined.

2 Experimental

2.1 Materials and Preparation

Table 1 indicates the property of the post-installed bonded anchors used in the experimental procedures. The inorganic-type post-installed bonded anchor contained ultra-rapid hardening cement, silica sand and additives in a cartridge. Figure 1 illustrates the usage of the inorganic-type anchor. First, the special water in a pack was poured into the cartridge, then mixed with a special mixer. Second, the cartridge was set at a caulking gun, and an anchor material was injected into a drilled hole. The water binder ratio at this anchor material was 0.41. The organic-type anchor mainly contained bisphenol A epoxy acrylate and peroxide mixed by a mixing nozzle and injected into a drilled hole.

Table 2 lists the concrete and deformed bar property. The compressive strength test of the concrete was based on JIS A1108. The specimen size was \( \phi 100 \times 200 \) cylinder. Figure 2 shows the specimen shape in the bond strength and creep tests. Concrete was placed into the steel pipe with a size of \( \phi 216 \times 91 \) mm to prevent the splitting cracks at the bond test. The deformed bar D13, which was a high-strength type bar, was connected to M14 (type: SNB7) by flash butt welding to join with a tension rod of the creep device. The embedment length was seven times the nominal diameter, such that no yielding occurs at an ultimate load. The drilled hole was made with a wet-type concrete core drill. The drilled hole at the inorganic- and organic-type anchors had a diameter of \( \phi 18 \) mm and \( \phi 16 \) mm, respectively. D13 penetrated into the specimen and settled in there. Bond strength tests were performed after more than 28 days of anchor installation. The creep test was started after which.
2.2 Bond strength test

The bond strength tests were conducted at an age of more than 3 months with less increase in strength during the creep test. Figure 3 shows the bond strength test apparatus. A reaction plate with an outer diameter of 180 mm and an inner diameter of 30 mm (Figure 4) was set on the specimen to produce a bond failure. The specimen was loaded using the centre hall jack. The load and the displacement of the free (δF, Figure 3) and loading (δL, Figure 3) ends were measured at this test.
The specimen was loaded using the centre hall jack. The load and the displacement of the free (\( \delta F \), Figure 3) and loading (\( \delta L \), Figure 3) ends were measured at this test. The measurement height at the loading end was 130 mm from the concrete surface.

### 2.3 Creep test

Figures 5 and 6 depict the creep test apparatus. A specimen was placed in the upper part of the apparatus, and a sustained load was applied with a coil spring. The bolt in the lower part of the apparatus was tightened after compressing the spring with a hydraulic jack. The specimen was subjected to sustained load when the load applied by the hydraulic jack was removed.
Table 3 lists the creep test conditions. Two types of anchor were prepared: inorganic- (InB) and organic-type (OrB) post-installed bonded anchors. The creep test was performed at 20 °C and 60% relative humidity. The reaction plate was positioned similar to that in the bond strength test.

Sustained loads were applied at 0.32 to 0.95 times the ultimate load to investigate their influence. The load and the displacement at the free (δF, Figure 6) and loading (δL, Figure 6) ends were measured in each test. The sustained load duration was more than 3 months.

### 3 Results

#### 3.1 Bond strength test

Table 4 and Figure 4 show the bond strength test results. The bond strengths ($\tau b_1$) were calculated as follows using Eq. (1):

$$\tau b_1 = \frac{P_{max}}{\pi \cdot da_1 \cdot lb} \quad (1)$$

- $\tau b_1$: bond strength
- $P_{max}$: ultimate load
- $da_1$: nominal diameter of the deformed bar
- $lb$: embedment length

The average bond strength of the InB was 25.7 N/mm$^2$, whereas that of the OrB was 22.6 N/mm$^2$. A defect was observed in one specimen in the OrB. Out of three specimens, one appeared to be of poor curing. Therefore, the bond strength was calculated as the average of two normal specimens.
Figure 6 shows the load–displacement curve at the bond strength test. The displacement at the ultimate load was 0.35 mm in the InB and 0.96 mm in the OrB. The displacement–load curve after the ultimate load was observed with a step-by-step downward trend in the InB, whilst that in the OrB gently decreased. The InB stiffness was higher than that of the OrB because the static elastic modulus of the InB was larger than that of the OrB.

### 3.2 Creep behaviour

Figures 8 and 9 show the creep curve at the free and loading ends. The creep displacement added the average value of the displacement of the sustained load at the bond strength test to the creep displacement after the sustained load was applied.

The creep failure of the InB occurred at a stress level (sustained load/ultimate load) of more than 0.79, whereas that of the OrB occurred at a stress level of more than 0.65 during the creep test. The time of the InB-0.79 creep failure was at 168 days, whereas that of the OrB-0.65 was at 15 h. In other words, the time of the OrB creep failure was rapidly faster than that of the InB at the same stress.

### Table 4: Results of the bond strength test

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Diameter</th>
<th>Ultimate load (kN)</th>
<th>Bond strength (N/mm²)</th>
<th>Displacement of the free end at ultimate load (mm)</th>
<th>Displacement of the loading end at ultimate load (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Anchor drilled hole length (mm)</td>
<td>Pmax Average</td>
<td>τb1 Average</td>
<td>δF Average</td>
<td>δL Average</td>
</tr>
<tr>
<td>InB</td>
<td>dₐ₁ dₐ₂ lₜ</td>
<td>102.2 89.0 93.5</td>
<td>28.2 24.5 25.7</td>
<td>0.48 0.27 0.35</td>
<td>1.47 0.87 1.14</td>
</tr>
<tr>
<td></td>
<td>① 12.7 18 91</td>
<td>89.2</td>
<td>0.30</td>
<td>1.08</td>
<td></td>
</tr>
<tr>
<td></td>
<td>②</td>
<td>81.9</td>
<td>22.6</td>
<td>0.94</td>
<td>1.45</td>
</tr>
<tr>
<td></td>
<td>③</td>
<td>82.1</td>
<td>22.6</td>
<td>0.98</td>
<td>1.23</td>
</tr>
<tr>
<td>OrB</td>
<td>① 12.7 16 91</td>
<td>82.0</td>
<td>22.6</td>
<td>0.96</td>
<td>1.46</td>
</tr>
</tbody>
</table>

Figure 7: Relationship between the load and the displacement at the free end
level. The failure mode at the creep failure happened at the interface between D13 and the anchor material.

Figure 8: Relationship between the duration of loading and the displacement at the free end

Figure 9: Relationship between the duration of loading and the displacement at the loading end

Figure 10: Relationship between the stress level and the displacement at 90 days
The displacement in the load–displacement curve at the bond strength test of the OrB increased from the point of approximately 70% of the ultimate load. For this behaviour, the possibility of the creep failure over this load appeared to be higher at the organic anchor.

The creep failure for the InB-0.95 with a high stress level occurred within 5 min from the application of the sustained load. The displacement of the free end just before the creep failure was almost equal to that in the bond strength test. However, the displacement of the InB at the creep failure after several hours from the application of the sustained load was 1.5 to 3.0 times larger than that at the ultimate load at the bond strength test (Figure 7). Similarly, the displacement at the creep failure at the loading end was 1.3 to 2.0 times larger than that at the ultimate load (Figure 8).

In InB, the longer the time at the creep failure, the greater is the displacement. In addition, the displacement at the creep failure was larger than that at the ultimate load of the bond strength test. Because displacement at the creep test is made the displacement which is added the displacement of creep deformation to the displacement of elastic deformation. Hence, the displacement at the creep failure was larger than at the time of the bond strength test. The number of data on the relationship between the time at the creep failure and the displacement at the creep failure was small. This relationship could be unclear, and further work is needed to confirm it.

Figure 10 shows the relationship between the stress level and the displacement at the age of 90 days, where the displacement after the sustained load application was set to 0 mm.

The slope of the OrB was larger than that of the InB. Accordingly, the difference of the material character was considered to affect the difference of the displacement for the same stress level.

Nakano et al. reported that the creep limit of the inorganic-type anchor was at the stress level of 0.6–0.7, whereas that of the organic type was at a stress level of approximately 0.5 [6]. The displacement of the stress level 0.6 of the InB and 0.5 of the OrB at the age of 90 days was 0.4–0.5 mm (Figure 10). The stress level was thought to be less than the creep limit, and creep failure does not occur, if the displacement at the age of 90 days was less than 0.4–0.5 mm.

### 3.3 Evaluation from the creep curve

Figure 11 shows a schematic diagram of the time to failure. Figure 12 illustrates a schematic diagram of the creep curve corresponding to the symbol in Figure 11. The creep limit was at the stress level, where creep failure does not occur (i.e. the point shown between C and D in Figure 11).

Eq. (2) with an exponential function was used with the creep prediction equation for the long-term creep deformation in EOTA ETAG [7].

\[
S(t) = S_0 + a \cdot t^b
\]  \hspace{1cm} (2)

- \(S_0\): initial displacement under the sustained load at \(t = 0\) (measured directly after applying the sustained load)
- \(a\) and \(b\): constants (tuning factor) evaluated by a regression analysis of the deformations measured during the sustained load tests
For the test method of the compressive creep test of concrete (Japanese Industrial Standard) [8], Eq. (3) with a logarithmic function was used as follows to obtain the long-term creep strain from the measured deformation:

$$S(t) = A \cdot \log_e(t + 1) + B$$

(3)

$A$ and $B$: constants (tuning factor) evaluated by a regression analysis of the deformations measured during the sustained load tests

Creep predictions were made to compare Eqs. (2) and (3) for the long term. Figures 13 and 14 show the results of the creep curve. Table 6 presents the result of the estimated equation.

The estimated displacement was extrapolated from the measured deformations using Eqs. (2) and (3). The displacement at 50 years was also calculated. The estimated results of the OrB-0.33 at the free and loading ends were presumed to be less likely to cause a creep failure because of being less than the average displacement at the ultimate load in the bond strength test.

Meanwhile, the displacement at the free end in the InB-0.32 was estimated to be 0.35 mm and 0.64 mm at the loading end and at 50 years, respectively. The estimated displacement of the OrB-0.33 at the free end was larger than the average displacement at the ultimate load in the bond strength test. However, the estimated displacement at the loading end was significantly smaller than the average displacement at the ultimate load. The reached time of the OrB-0.33 to the displacement at the ultimate load greatly differed between the free and loading ends. This was why the displacement of the free end at the bond strength test was very small, and a slight difference (e.g. variations in the test specimen and error of the estimated equation) affected the long-term estimated creep deformation.

The safety evaluation of the long-term creep confirmed that the predicted displacement was less than that at the strength test.

The displacement at the creep failure was larger than that in the bond strength test. Therefore, the evaluation with the displacement at the bond strength will be a safer-side evaluation. However, the

![Figure 11: Schematic of time to failure](image1)

![Figure 12: Schematic of the creep curve](image2)
The estimated displacement from Eq. (2) at 50 years was calculated as a value approximately 1.3 times as large as the estimated one from Eq. (3). A large difference was found in the estimated result depending on the applied equation for the long term. Hence, applying a suitable equation for the materials was necessary.

3.4 Evaluation from time to failure

Figure 15 shows the time to failure. The bond strength test results were plotted in this figure assuming 3 min at the bond strength test. The results of Eqs. (2) and (3) were also added in Figure 15, in which the time to failure was calculated from the creep limit displacement at 90 days. The InB slope of time to failure from the creep failure results was approximately 1/3 of OrB. The estimated line slope of Eq. (2) was equal to the time to failure from the creep failure results in InB.

Meanwhile, the estimated line slope of Eq. (3) was equal to the time to failure from the creep failure results in OrB.

Figure 13: Relationship between the duration of the loading and the displacement at the free end

Figure 14: Relationship between the duration of the loading and the displacement at the loading end
Form this point, the displacement estimation from the creep curve was assumed to be applied to the inorganic injection-type anchor, and the exponential function was applied to the organic anchor.

4 Conclusion

This study involved the conduction of creep tests on post-bonded anchors. The deformation under a sustained load was measured at the free and loading ends. The long-term creep deformation was estimated using the exponential and logarithmic functions from the measured deformation. Their applicability was also examined.

1) The evaluation value from the free and loading ends greatly differed in the creep failure estimation from the creep curve of the inorganic-type post-installed anchor because the displacement during the bond strength test was small. Consequently, a slight difference greatly affected the estimated time of creep failure occurrence because of the variations in the specimen and errors in the estimated equation.

Table 6: Results of the estimated deformation

<table>
<thead>
<tr>
<th></th>
<th>Estimate from the free end</th>
<th>Estimate from the loading end</th>
</tr>
</thead>
<tbody>
<tr>
<td>InB-0.32</td>
<td>Eq. (2) ( S(t) = 0.055 + 0.0244t^{0.255} )</td>
<td>Eq. (3) ( S(t) = 0.335 + 0.0323t^{0.232} )</td>
</tr>
<tr>
<td>InB-0.63</td>
<td>Eq. (2) ( S(t) = 0.135 + 0.1240t^{0.261} )</td>
<td>Eq. (3) ( S(t) = 0.628 + 0.132t^{0.253} )</td>
</tr>
<tr>
<td>OrB-0.33</td>
<td>Eq. (2) ( S(t) = 0.084 + 0.0753t^{0.189} )</td>
<td>Eq. (3) ( S(t) = 0.231 + 0.0731t^{0.193} )</td>
</tr>
<tr>
<td>OrB-0.50</td>
<td>Eq. (2) ( S(t) = 0.126 + 0.1452t^{0.209} )</td>
<td>Eq. (3) ( S(t) = 0.331 + 0.152t^{0.205} )</td>
</tr>
<tr>
<td>OrB-0.57</td>
<td>Eq. (2) ( S(t) = 0.154 + 0.229t^{0.298} )</td>
<td>Eq. (3) ( S(t) = 0.381 + 0.232t^{0.299} )</td>
</tr>
</tbody>
</table>

Figure 15: Time to failure calculated from the displacement of the estimated creep failure
2) The value estimated by the exponential function when the creep deformation at the age of 50 years was estimated from the measured creep curve by the exponential and logarithmic functions was approximately 1.3 times larger than that by the logarithmic function.

3) The estimation of the displacement from the measured creep curve was applied to the inorganic injection-type anchor. The exponential function was then applied to the organic anchor.

References:


8. JIS A 1157: Method of test for compressive creep of concrete, Japanese Industrial Standards, Japan, 2010
BOND CAPACITY OF ADHESIVE ANCHORS IN BOREHOLES WITH CAVITIES

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ABSTRACT

Adhesive anchor systems are widely used for the fastening of threaded rods and reinforcing bars in concrete. These systems offer a wide variability in terms of the installation parameters (embedment depth, member thickness, minimum distances), admissible service conditions (e.g. temperature of installation and use), drilling method and the bearing capacity.

In a special application the aim of the research was to find a solution for fixing façade elements in a concrete wall with cavities. Therefore, the load bearing behavior of adhesive anchors fastened in concrete bore holes with cavities was investigated. The position of the cavity in the concrete wall varied so that the bond length above and below the cavity in the borehole was different. Tension tests with confined and unconfined test setup and different bond length were performed to determine the bond capacity and the concrete cone breakout capacity considering the existing cavities.

The paper describes the special application and installation of anchors in boreholes with cavities. The results of tension tests are presented and a load bearing model is discussed.

1 Introduction

For a building it was planned to fix a façade system to the wall using anchors with baseplates. However, a solution has to be sought to find a fixing system that works properly in concrete walls with cavities. The cavities are present due to a special concreting or continuous concreting process after the formwork is removed or because some unused pips are present.

In the described application the cavities are round with a diameter of approximately up to 52 mm. The position of the cavities was about 80 mm to 120 mm behind the concrete surface. The cavities were distributed more or less arbitrary in the wall. Due to the complex shape of the wall they could not be detected easily and properly enough to avoid an anchorage in this area. Due to a high demand of loads\textsuperscript{6}, a mechanical system with an embedment depth of less than $h_{ef} = 80$ mm was also not suitable. Therefore, a bonded injection system of fischer\textsuperscript{2, 3} with a larger embedment depth of 180 mm was used. The system was an epoxy based bonded injection system with threaded rod M16 with a valid assessment\textsuperscript{2, 3} for the use in uncracked and cracked concrete. Since the embedment depth was larger than the “concrete cover” over the cavities the risk to drill a cavity or that a cavity is
nearby the area of the anchorage zone was quite high. But the system has two advantages compared to mechanical systems, especially mechanical expansions systems.

(i) The first advantage is that the system provides high loads and can be used with flexible embedment depths up to 20d. The required embedment depth was about 100 mm to 130 mm in areas were no cavity is present. The allowable embedment depth of the chosen system was 64 mm to 320 mm. Thus the bonded injection system is suitable in both areas with and without existing cavities because the embedment depth can be increased up to 200 mm if required. This option was chosen in areas were cavities are present.

(ii) The second advantage of bonded anchors is that the expansion forces are very low. For expansion anchors the risk of a local damage in the area of the expansion clip is very high if a cavity is present close to the borehole. The expansion forces would result in a crack that runs in the direction of the cavity. This risk is minimized for bonded anchors due to the low and well distributed expansion forces.

To be able to use a bonded injection system in concrete walls with cavities the installation with a perforated sleeve is obligatory. Otherwise the adhesive disappears in the cavity. Bonded anchor systems with perforated sleeves are widely accepted in perforated brick material. Generally, no perforated sleeve is used for bonded injection systems that are used for concrete only. If the system has an assessment for the use in perforated brick material the system at least may provide the necessary components for an installation with a perforated sleeve. Due to the fact that the intended epoxy system was not approved in perforated anchorage ground, experiments were performed to answer the following questions:

1. Can the bonded injection system be used with a perforated sleeve in concrete? Can the system be installed properly in concrete with and without cavities up to an embedment depth of 200 mm?
2. Is the mean bond strength comparable and how is the bond strength influenced, if the anchor is installed close to a cavity?
3. How do the cavities influence the bond strength if the anchor is installed in or through a cavity?

2 Scope

The scope of the research was to compare the results of the tests of different configurations of bore holes with cavities (see figure 1b, c) with the reference configuration of a bonded anchor installed in concrete without any influence of cavities and without perforated sleeve (see figure 1a).

The aim of the tests was to understand the influence on the mean bond strength of the modified system (application with perforated sleeve) by a direct comparison of the results for the “original” approved system (application without perforated sleeve). The results are assessed to derive a design model for the calculation of the tension resistance in relation to an effective bond length $h_{\text{eff}}$ and bond strength $\tau_{\text{Rk}}$. This model should take into account the different configurations (cavity to borehole).
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In principle there are three different configurations that were assessed. The first configuration corresponds to the reference configurations where no influence of cavities is present (Figure 1a). For the second configuration the bond length is reduced due to the fact that a cavity is present in the area of the anchorage zone (Figure 1-b). In the third configuration the cavity is close to the anchorage zone and may change the stress distribution and consequently the tension resistance in an unfavorable way (Figure 1c).

The results are assessed in a way that the mean bond strength and the load-displacement behavior are compared for the given configurations to see if there are significant differences.

![Figure 1: Possible configurations of cavities in the area of the anchorage zone of a bonded anchor.](image)

First an assessment of the installation procedure and subsequent the assessment of the capacity of the different systems was conducted and is described in the following sections.

### 3 Experimental investigations

**3.1 Installation tests**

At the beginning installation tests were performed to see whether the system can be used with a perforated sleeve. Therefore, the system was installed in a concrete cube with a cavity that is truncated by the borehole. The cavity was produced using diamond core drilling with a diameter of 50 mm. The bore hole was drilled in a way that the bore hole fully truncates the cavity centric and perpendicular.

Then the threaded rods and the perforated sleeves are cut to the correct necessary length. The perforated sleeve was pushed into the bore hole and the adhesive was injected. Then the threaded rod was pushed into the adhesive filled perforated sleeve. After a curing time of at least 19h (at a temperature of 20°C) a special washer and nut was sampled on the top to fill the remaining gap of the fixture and the upper part of the anchor. After that the anchor could cure properly.
After the curing the concrete cubes were split in two parts to see if the injection was properly and without air voids. The results show that if the gap is filled with adhesive using the special washer no voids are visible in the member and the bond to the concrete surface was completely developed. The installation tests were performed at three different temperatures, 5°C, 20°C and 40°C to see if the viscosity at low and high temperatures influences the installation. In Figure 3 the results for the 5°C tests are shown 1.

Consequently, it was established that the system can be used with a perforated sleeve in concrete and that the system can be installed if a borehole truncates a cavity. The described installation procedure is sufficient to ensure a proper filling of the borehole with adhesive and to exclude voids in the area of bond transfer.

### 3.2 Performance tests

In tension tests with unconfined setup three different configurations (see figure 1) were tested to determine how the bond strength is influenced by the presence of cavities. In all tests the nominal embedment depth was $h_{ef} = 180$ mm. The diameter of the cavity was 52 mm. Thus the minimum
effective bond length was $h_{e,_{eff}} = h_e - 52 \text{ mm} = 128 \text{ mm}$ for the configurations given in figure 1-b1) and b2).

The results of the mean bond strength $\tau_{um}$ observed for the different tested configurations show that the mean bond strength is between 15 N/mm² and 20 N/mm². The bond strength calculated with $h_{e,_{eff}}$ (Equation 2) is independent of the tested location or position of the cavity.

The results show that for the reference tests a mean failure load of 161,7 kN was achieved. For the tests where the borehole totally truncate the cavity ($d = 52 \text{ mm}$) the mean failure load is less. In this case the measured mean failure load was between 113,5 kN and 119,8 kN depending on the location of the cavity. However, all bond strength derived with the effective bond length $h_{e,_{eff}}$ are within the expected scatter for the base material\(^1\).

If the borehole does not truncate the cavity but is nearby, the mean failure load drops down to roughly 90% of the reference value. This may be due to an unfavorable stress distribution in the anchorage zone because of a notch effect.

### Table 1: Tested configurations and test results.

<table>
<thead>
<tr>
<th>Tested configurations</th>
<th>[-]</th>
<th>Figure 1 - a)</th>
<th>Figure 1 - b1)</th>
<th>Figure 1 – b2)</th>
<th>Figure 1 – c)</th>
<th>Figure 1 – c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete cover from the concrete surface to the cavity [mm]</td>
<td></td>
<td></td>
<td></td>
<td>80</td>
<td>108</td>
<td>80</td>
</tr>
<tr>
<td>Bond length $h_{e,_{eff}}$ [mm]</td>
<td>184,3</td>
<td>130,4</td>
<td>131,7</td>
<td>187,7</td>
<td>178,8</td>
<td></td>
</tr>
<tr>
<td>Mean failure load [kN]</td>
<td>161,7</td>
<td>113,5</td>
<td>119,8</td>
<td>150,9</td>
<td>145,8</td>
<td></td>
</tr>
<tr>
<td>Mean bond strength [N/mm²]</td>
<td>17,4</td>
<td>17,3</td>
<td>18,1</td>
<td>16,0</td>
<td>16,2</td>
<td></td>
</tr>
</tbody>
</table>

If the failure loads and the effective bond length $h_{e,_{eff}}$ (Equation (2)) are taken to calculate the mean bond strength the results can be given as a function of the effective bond length. It is obvious that the bond strength does not depend on the existing bond strength nor if a cavity is present. The measured bond strengths are between 15 N/mm² and 20 N/mm² (see Figure 4) for all configurations illustrated in figure 1.

Also the scatter of the failure loads of the single test series was comparable and the failure mode didn’t change for the different configurations. In figure 5 the failure modes for the different configurations are displayed. In all cases a pullout failure with a concrete cone close to the surface could be observed. In all tests radial cracks starting from the anchors are visible at peak load. This can be clearly seen in figure 5a where the configuration according to figure 1a was tested.

The typical failure mode of a combined pullout with concrete cone failure that was observed in the tests can be seen in figure 5b. The present cavity nearby the borehole of the bonded anchor can be seen in figure 5c and figure 5d (detail of figure 5b). The results show that the cone and the cracking runs into the cavity that is located close to the anchorage zone\(^1\).
Figure 4: Measured bond strength in tension tests (unconfined test setup) as a function of the effective bond length \( h_{\text{ef,eff}} \) (Equation (2)) for the configurations displayed in figure 1.

All in all the results show that the system can be properly installed and that the bond strength calculated with the effective bond length is nearly not influenced if the borehole truncates a cavity and if a perforated sleeve is used for the tested bonded injection system.

4 Conclusions and Outlook

The test results show, that existing cavities with a diameter up to about 50 mm in a concrete member do not influence the mean ultimate bond strength taking into account the effective bond length \( h_{\text{ef,eff}} \) of the tested bonded injection system. This is valid for the tested configurations independent of the cavity position in relation to the anchorage. A precondition is that the system (i) can be properly installed with a perforated sleeve and (ii) the bond strength is not influenced by the perforated sleeve. Both conditions must be ensured using comparable tests were the system is tested as approved (reference tests without perforated sleeve) and tests under the same condition but using a perforated sleeve in addition (reference tests with perforated sleeve). The tests were performed at three temperatures, 5°C (minimum installation temperature), 20°C (reference temperature) and 40°C (maximum installation temperature).

Furthermore, the tests result show that cavities nearby the anchorage zone will slightly negatively influence the combined pullout and concrete cone capacity. To see if this effect is significant more tests should be performed or a reduction factor of 0.85 should be taken into account.
Based on the test results it is proposed to calculate the characteristic resistance of bonded injection anchors according to Equation (1) by using the characteristic bond strength given in the approval and the effective bond length as embedment depth:

$$N_{Rk,p}^0 = \psi_{Cav} \cdot \tau_{Rk}^0 \cdot \pi \cdot d \cdot h_{ef,eff}$$  \hspace{1cm} (1)

with:

- $\psi_{Cav}$ = Reduction due to local stress distribution of cavities nearby the anchorage zone
- $\tau_{Rk}^0$ = Characteristic bond strength given in the approval
- $d$ = Diameter of the anchor
- $h_{ef,eff} = h_{ef} - d_{Cavity}$
- $\tau_{Rk}^0$ = Reference bond strength of the bonded anchors if a perforated sleeve is used in concrete

$$h_{ef,eff} = h_{ef} - d_{Cavity}$$  \hspace{1cm} (2)
In equation (1) the effective embedment depth (bonded length) is calculated using the embedment depth of the system ($h_{ef}$) and then subtracting the diameter of the cavity ($d_{Cav}$). For the tested system this means that the embedment depth is reduced from $h_{ef} = 180$ mm to $h_{ef,eff} = 128$ mm due to cavity diameter of $d_{Cav} = 52$ mm. Equation (1) is only valid for the tested combinations if the distance of the cavities is large, so that only one cavity is within the anchorage zone of one anchor or an anchor group.

If a group of anchors is installed in the area where a cavity is present, some anchors of the group may be influenced by the cavity and the others may be not influenced by the cavity. Thus the effective embedment depth $h_{ef,eff}$ of the anchors in the group may different. To determine the effect on the group behavior, the stiffness’s and the load redistribution must be taken into account. This should be taken into account for anchor groups.

To simplify the design, the easiest way is to limit the embedment depth to the effective embedment depth of the anchors truncating the cavity. This assumption is conservative.

Other effects and reductions like the effect of sustained loading or the influence of temperature are taken from the assessment document of the anchor. In absence of information the DIN EN 1992-4\textsuperscript{8} or EAD 330232-00-0601 provides some standard values that can be taken into account for the design.

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EFFECT OF DECREASED INSTALLATION TEMPERATURES ON THE LOAD BEARING BEHAVIOR OF ADHESIVE ANCHORS

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ABSTRACT

Adhesive anchors are versatile and often used in reinforced concrete structures to connect structural and non-structural components. Particularly when used for exterior applications, their major advantage is that the adhesive seals the borehole and prevents the penetration of moisture and other destructive substances into the base material. At the same time, however, adhesives can be exposed to strong temperature variations. This results in load and creep resistance behaviors that differ from those of adhesive anchors in indoor applications under nearly constant ambient temperature conditions.

Therefore, temperature effects occurring during the service life of adhesive anchor connections are considered in the prequalification procedures of ETAG 001, P.5, ACI 355.4-11 and ICC-ES AC308. The prequalification tests include short-term tests at different temperatures as well as tests under long-term load at ambient temperature and application-related long term temperature. Sustained load tests at minimum temperature were not required in the past because it was assumed, as with polymers in general, that the tendency of adhesives to creep decreases with temperature.

Until recently, the behavior of adhesive anchors installed in cold concrete components was inadequately investigated. This contribution provides detailed background information on the effect of low temperature installation of adhesive anchors on their load bearing behavior in service, describe corresponding tests and their results, which finally resulted in the prequalification procedure of ICC-ES AC308. Based on this information the actual prequalification procedures for adhesive anchors are discussed.

1 Introduction

In the last years tremendous progress has been achieved in enhancing the fields of applications for innovative adhesive anchor systems. Therefore it also became more and more common to install adhesive anchors in concrete also during winter i.e. in cold weather conditions where the temperature of the base material governing the behavior of the adhesive anchor is also cold to ensure safe and economical construction processes (Fig. 1).

The final load-bearing behavior of adhesive anchors is significantly influenced by the curing conditions of the chemical mortar. At low temperatures the curing rate is low and glass transition
temperature, bond strength and creep resistance of the cured adhesive are reduced compared to the values obtained after a curing process at higher temperatures.

In the following results of tests and the corresponding evaluations for chemical mortars used for adhesive anchors and adhesive anchors are presented. With these findings the contents of the actual prequalification procedures for adhesive anchors are compared. More details on the investigations and their results are given in a paper published in Concrete International in January 2016.

Figure 1: Injecting adhesive mortar at low base material temperature

2 Chemical mortars

2.1 General

Chemical mortars used for adhesive anchors are generally based on thermosetting adhesives such as vinylester (epoxy acrylate) or epoxy resins and their corresponding hardeners. The curing (hardening) of vinylester resins occurs relatively fast by means of a polymerization, the reaction of epoxy resins (slow cure adhesives) is generally based on the relatively slow poly-addition reaction mechanism. After curing according to the manufacturers’ product installation instructions, chemical mortars indicate sufficient bond strength to reliably and permanently transferring high loads from the attachment into the concrete structure.

If the reaction process runs at low temperatures, the degree of chemical conversion i.e. the cross-linking of the polymer chains can be expected to be lower than the degree of conversion of the same adhesive cured at higher temperatures over the same time period. Less cross-linking yields lower adhesive strength, an increased tendency for creep deformations, and higher sensitivity to temperature changes. After installation complete cross-linking is rarely achieved under on site conditions. Since the curing process of epoxy resins is substantially more sensitive to the environmental temperature than the curing process of vinylesters and the effects of low temperature can be better visualized in the following only the investigations with an epoxy system are reported.
2.2 Softening behavior

As a result of temperature increase the material characteristics of the adhesive mortar are unfavorably influenced, the chemical mortar may soften and failure of the adhesive anchor due to extensive creep might occur. In practice this temperature increase can start after installation at low temperature in presence of direct sun exposure.

A first indication for the softening is the glass transition temperature $T_g$ at which the hardened resin transitions from a hard and glassy to a soft material. However, it has to be pointed out that the softening of the material which is critical for engineering applications starts approximately 10°C to 20°C below $T_g$, (Fig. 2). Therefore the glass transition temperature $T_g$ has no direct meaning to applications in structural engineering practice.

The German Standard DIN 65583 gives rules on the determination of the softening temperature. It is defined as the temperature at which the storage modulus has decreased in a dynamic-mechanical analysis (DMA) by about 2% compared to a temperature-dependent continuous decrease. In this manner, a sufficient distance from the softening point of the synthetic resin can be maintained in practice.

![Figure 2: Determination of the softening point according to DIN 65583](image)

2.3 Investigations

Laboratory tests of the chemical mortars alone do not reflect conditions on site. Major parameters influencing the bond strength of adhesive anchors such as the interaction of the chemical mortar with the borehole wall as well as effects originating from the installation process are not considered. In case of the determination of the degree of curing and its sensitivity to temperature, however, relevant results can be achieved.

The degree of cure and $T_g$ of the epoxy mortar were determined from tensile test bars according to ISO 31674 by means of differential scanning calorimetry (DSC) according to ISO 11357. In the context of these investigations the test specimens were cured at temperatures of 5°C, 23°C, and 43°C, for a period of 150 hours. Subsequent to curing, the test specimens were exposed to different temperature regimes. The results of the developed glass transition temperatures $T_g$ are summarized in Figure 3.
Figure 3: Development of $T_g$ as function of time and initial curing temperature

Figure 3 indicates that the glass transition temperature $T_g$ rises significantly with the temperature at initial curing. However, only a slight increase of $T_g$ can be observed with curing continued over 1000h at the initial curing temperature. The same $T_g$ as curing at a constant temperature of 23°C or 43°C is detected if the tests are cured at low temperatures and immediately after initial curing heated to 23°C or 43°C. This means that the reaction mechanism of the tested adhesive can be significantly accelerated by increasing the temperature and the cross linking of the chains is not unfavorably influenced by a low installation temperature.

The degree of cross linking i.e. the degree of curing is shown in Figure 4 for the before mentioned tests. The glass transition temperature $T_g$ increases with the degree of cross linking. The comparison at a curing temperature of 43°C yields after 150h and 1150h the nearly the same degree of cross linking. A maximum degree of curing of about 97% could be achieved. This implies that temperature increase initiated post-curing occurs and the degree of curing is nearly independent of the curing time.

Figure 4: $T_g$ as a function of degree of curing (cross linking)
In the following it is checked if the results of the chemical mortar tests can be reproduced in the load bearing behavior of adhesive anchors.

3 Adhesive anchors

3.1 General

Adhesive anchors were tested confined under tension to ensure bond failure in short term pull-out and sustained load tests with 12mm anchor rods installed properly according to the manufacturers’ product installation instruction in boreholes of 14mm diameter and 80mm embedment depth in non-cracked low strength concrete \( (f_c \approx 25\text{N/mm}^2) \) with test setups according to EOTA ETAG, P.5 specifications.

3.2 Short term tests

All short term tests were installed and cured at a temperature of 5°C. The first test series was performed directly after the minimum cure time of 50h was completed. The next series was performed after heating of the specimen from 5°C to 23°C within 24h and the last series after an additional heating to 43°C within further 24h. The bond-slip curves are shown in Figure 5.

Figure 5 shows that, when cured at low temperatures, the failure mode of the epoxy adhesive anchor system changed in the short-term tests from a failure between chemical mortar and threaded rod to a rupture between chemical mortar and bore hole wall. In addition a reduction of the peak load was measured after a temperature increase to 43°C. This is caused by the slow post-curing occurring after the initial curing at 5°C. Then the possible crosslinking of chains i.e. a vitrification of the adhesive at this temperature level was completed. During subsequent heating over 48h to 43°C the development of new cross-links was not fast enough to ensure a solid polymer and softening due to the rising temperature could be observed. After holding the temperature at the elevated level the reaction process ran until the possible cross-links at this temperature level were formed and a revitrification occurred. This effect is shown in Figure 6 and agrees with the findings of the tests with the epoxy resin specimens (see Figures 3 and 4).
3.3 Sustained load tests

The sustained load tests were performed at service load level i.e. at a lower load level as required in prequalification tests to get an idea on the safety level of existing adhesive anchor connections installed under low temperature conditions.

The adhesive anchors were subjected to the sustained tension load immediately after completing the curing time according to the manufacturer’s installation instructions. During the initial 150 hours of sustained load, the specimens were maintained at the installation temperature. This was done to determine the effects of the curing temperature on the degree of cure by means of measuring possible increase of displacement. Then, depending on the test series, the temperature remained at 5°C or 23°C for the duration of the sustained load test or the temperature was raised from 5°C at time of installation and sustained loading over 150h to 23 or 43°C within 72 hours and the test continued to the end.

The adhesive anchors installed, cured, and tested at a consistent low temperature of 5°C or 23°C showed both only minimum slip increase for the full duration of the sustained load test. Therefore, the tested epoxy resin system is also suitable for use in applications with sustained loads at consistently low temperature conditions.

The test series which cured at 5°C for 50h and carried a sustained load for another 150h at this temperature level also showed nearly no increase of anchor displacement. This is shown in Figure 7 which gives plots of displacement and temperature versus time curves for the tests which were heated from 5°C to 43°C. Figure 7 also indicates that after the beginning of the warming with a small delay a considerable raise of displacement starts. This increase clearly decelerates with time because the temperature rise accelerates the reaction of the epoxy resin and leads to a higher degree of curing i.e. cross-linking. Figure 8 clearly demonstrates this effect. After a strong increase, the speed of the displacement decreases rapidly. Similar results were found in the test series which were heated from 5°C to 43°C\(^8\). This behavior agrees with the knowledge gained in the investigations with the epoxy test specimens (Fig. 5 and 6).
Figure 7: Displacement versus time curves measured in the test series which were heated from 5°C to 43°C

Figure 8: Creep rate measured in the test series which were heated from 5°C to 43°C

4 Prequalification procedures

In Europe, the approval guideline ETAG 001, P.5 stipulates only confined short term tests to check the influence of installation and subsequent load-bearing behavior of adhesive anchors. Sustained load tests to check the influence of heating and possible softening of the adhesive mortar are not requested.
The US prequalification procedures ACI 355.4-11\textsuperscript{9} and ICC-ES AC308\textsuperscript{10} require the verification specific to the low installation temperature for adhesive anchors that are prequalified for installation into a concrete component with a temperature below 10°C (50°F). For a minimum installation temperature between 5°C and 10°C both prequalification procedures require confined short term tests at the intended temperature. For minimum installation temperatures lower than 5°C the adhesive anchor is installed and cured at the intended minimum installation temperature level, then subjected to a sustained load and immediately after loading heated under sustained load for 72h to 96h to 23°C. This represents a very slow heating rate of only 0,25 to 0,5K/h. Finally a residual load test is performed. This procedure, however, does not consider the effect of fast temperature rise as it might occur for example during façade installations in spring where solar radiation can lead to subsequent fast heating of the connection. Therefore in ICC-ES AC 308 an additional optional test with a heating rate of 5K/h representing temperature recordings in concrete structures\textsuperscript{11} was introduced. The test procedure is summarized in Figure 9.

The adhesive is cured at minimum installation temperature according to the manufacturers’ product installation instruction, subjected to a sustained load, heated in presence of the sustained load with 5K/h up to 23°C and remains at this load and temperature level for at least 1000 hours. The measured additional displacement due to possible softening compared with the sustained load test at normal temperature is used for the assessment of the behavior. The total displacement extrapolated from the sustained load tests to a duration of 50 years shall not be allowed to exceed a limiting value developed from the load-displacement characteristics of the short-term reference tests, where the adhesive anchors failed in bond.

5 Conclusion

The influence of a low-installation temperature on the load-bearing behavior of adhesive anchors with a subsequent heating e.g. resulting from solar radiation in façade applications, is described in this paper for an epoxy resin. The same effects can be found in a diminished form also on vinylesters\textsuperscript{8}. While in Europe the effect of possible softening of the adhesive is not considered in the
ETAG, P.5 prequalification procedures, ICC-ES AC308 provides an optional test based on the before mentioned investigations to overcome this problem and to yield safe and economical fastenings with adhesive anchors even after installation in winter or early spring.

It is highly recommended to supplement the European prequalification guidelines for adhesive anchors to consider installation at low temperature effects.

6 Acknowledgement

The authors would like to thank the Concrete and Masonry Anchor Manufacturers Association (CAMA) for the financial support of the investigations.

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CHEMICAL FASTENING SYSTEMS GO GREEN — A CONTRIBUTION TO MORE SUSTAINABILITY

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ABSTRACT

"Green Building" or "Sustainable Building" have been catchwords in the building industry in recent years. In a lot of countries, rating systems for "Green Building" have been developed and the concepts enjoy increasing attention and importance. The various rating systems take complying building products into account in their rating processes, which inversely imposes an increasing pressure on producers to fulfill the required sustainability rules.

fischer as a leading supplier of innovative fastening systems has tackled these tasks and has developed the world’s first green chemical fastening system FIS Green. The top innovation product employs predominantly renewable raw materials. More than 50% of the organic raw materials used in the system are from rapidly renewable resources and are not based on fossil fuels. Furthermore FIS Green has very low emissions during application already and throughout its lifetime. It fulfills all VOC regulations and is a label-free system. While being green and sustainable, FIS Green is still a heavy duty anchoring system and no compromise has been made when it comes to performance.

With FIS Green, fischer has delivered the proof that it is possible to develop even complex multi-constituent products on a predominantly renewable basis without loss of performance.

1 Introduction

The tremendous wealth of our modern societies is mainly based on the exploitation of fossil resources like crude oil, coal and gas. For the most part, these resources are combusted and used as an energy source, although they are even more valuable and important to mankind as a carbon source. Modern life would not be possible without the conversion of fossil carbon to organic chemicals, pharmaceuticals and polymers. However fossil resources are finite, therefore new (renewable) carbon sources and strategies for their conversion into valuable goods will be needed urgently in the future.

Today sustainability is often only used as a fashionable buzzword and is a mainstream trend these days. But what is a trend today will be a sheer necessity tomorrow. The finiteness of fossil raw materials will enforce a transformation and there will be an urgent need in the future to base our economy on renewable and sustainable raw material sources.
2 Sustainability in the Building Industry

Throughout the world, efforts have been made to impose sustainability principles in the building industry as well. In a lot of countries, rating systems for "Green Building" have been developed, e.g. LEED for the US and Canada, the British BREEAM and the German DGNB. Usually, the rating systems evaluate the whole building from cradle to grave, not (as often erroneously assumed) individual products which are used to build them. The concepts of the rating systems vary and emphasize different aspects which are covered under the terms "Green Building" and "Sustainability", among others reduction of the environmental impact and the protection of inhabitants from harmful emissions evolving from building products. Products which comply with the corresponding rules are taken into consideration in the rating process. The application of such products in the construction of a building can for example earn credits for the rating of the building. This can and will be a driver in the future to induce the development of more sustainable products.

3 Chemical Fastening Systems

In the field of heavy duty connections between steel and concrete, the application of chemical fastening systems belongs to the methods of first choice. Chemical fastening systems allow the anchoring of steel elements in concrete free of expansion pressure. Thus, not only highest load bearing capacities, but also minimum edge and space distances are feasible. Over the last decades, chemical fastening systems have found an increasing acceptance in the market and are in widespread use today.

Chemical fastening systems are made up of two components. The first component contains a curable reactive resin, the second component contains the corresponding hardener. In most cases, the two components are placed in a two-component cartridge. The system is applied by screwing a static mixer onto the nozzle of the cartridge and pressing out the two components by means of a dispensing gun (Figure 1).

Figure 1: Preparation of a two-component chemical fastening system for its application.

In the static mixer, the two components are thoroughly mixed and the homogeneous mortar can be injected directly into the prepared and cleaned drill hole. The fastening element made of steel can be inserted into the mortar-filled hole manually. After the curing time of the mortar, the anchoring rod is securely fastened and the load can be applied. The fastening procedure is depicted in Figure 2.
4 Task

fischer has been a trailblazer when it comes to environmental, health and safety topics in connection with chemical fastening systems. We were the first to introduce styrene free systems in the market with negligible emissions for the protection of craftsmen, inhabitants and the environment. Since 1996, we have banned styrene completely in our product range. With that new generation of products, rock-bottom values for achievable VOC contents were reached and set as a new standard.

With all the aforementioned considerations, the logical consequence for a further advancement in the technology of chemical fastening systems was the formulation of the task: The development of a chemical fastening system made from a majority of bio-based renewable raw materials.

5 Solving the Task

All chemical fastening systems currently in the market are based on fossil raw materials. Sustainability or renewability has never been a topic so far although the task appears to be quite obvious in hindsight.

One reason the task had not been solved before may have been the fact that chemical fastening systems are very complex. The formulations contain a large number of different raw materials making it difficult to set a system on a new raw material basis.

But it is not a problem of a lack of bio-based raw materials in principle. In terms of sustainability, the chemical industry has proven to be quite far-sighted and has left the starting holes already. These days, an increasing number of renewably sourced substances is entering the market. And as the raw material base is getting broader, more and more sustainable products appear. Most of these products are quite simple, they often consist of a single raw material. In these cases, a simple drop-in approach is preferred, i.e. a fossil based raw material is exchanged by its identical renewable analogue (e.g. polyethylene plastic bag vs. bio-polyethylene plastic bag). It was clear from the start that a simple drop-in approach was doomed to fail for chemical fastening systems due to their complexity and the fact that most of the ingredients are simply not available in a bio-based version. So there was no perspective of reaching the task by just replacing one or two petro-based ingredients by their bio-based counterparts. The only way to reach the set goal was the reassessment of the whole product structure with a focus on sustainability. By this comprehensive approach, every single ingredient was scrutinized and evaluated for possible bio-based alternatives, and even completely new raw materials were taken into consideration. As a result, FIS Green is made up of a lot of different bio-based raw materials with various functions in the system:
- reactive resins and diluents for curing
- fillers
- additives
- plasticizers
- two different thermoplastic polymers as cartridge material and for the pistons

This radical and consequent product redesign approach made it possible to achieve a high percentage of renewable ingredients. FIS Green has been certified by DIN CERTCO as an independent expert to have a high bio carbon content of 50-85%. The bio carbon content can be determined by radiocarbon measurements according to ASTM 6866. The certification label is shown in Figure 3. The high bio carbon content demonstrates the far-reaching replacement of petrochemical raw materials by renewable resources.

![Figure 3: DIN CERTCO label certifying the content of bio carbon.](image)

6 Achievements

Some bio-based consumer products might be sustainable, but can not reach the performance level of their petrochemical counterparts. However, for a heavy duty fastening system with security relevance, a compromise in product performance was not acceptable.

Figure 4 shows the test results of confined pull-out test of M12 threaded rods in C20/25 concrete (drill hole 14x95 mm, cleaned by 2 times blowing, 2 times brushing and 2 times blowing). It can be seen that FIS Green reaches a good performance level and even outperforms a comparable petro-based reference system. FIS Green has an ETA approval for the application in concrete and masonry.
The product is label-free, does not contain any volatiles and has therefore a very low VOC content making it comply with the French VOC regulation (Figure 5). It was also the first chemical fastening system to earn a German Environmental Product Declaration (EPD). The label is depicted in Figure 6.

The special two component cartridge, which consists of a bio-polymer, makes it possible to use the system without special equipment by means of a simple dispensing gun for sealants. The cartridge is shown in Figure 7.
The successful development of a product like FIS Green is an impressive proof for the already existing large pool of bio-based raw materials and the possibility to re-formulate even complex products to make them bio-based and more sustainable. The holistic approach for the redesign of the complete system including the cartridge material, the resins and even the fillers was the key to success in developing a sustainable chemical anchoring system.

FIS Green is therefore a good contribution to more sustainability in the building industry. As its features are still unique, this contribution is currently not adequately honoured by "Green Building" rating systems. Here, new rules and possibilities to differentiate from standard products (e.g. by earning extra credits) would be desirable in order to further promote the development and use of sustainable products.
ANCHOR CHANNELS
COMPARISON OF DESIGN RULES FOR ANCHOR CHANNELS WITH CHANNEL BOLTS

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ABSTRACT

Anchor channels are cast in concrete and allow the connection of steel fixtures with matching channel bolts after stripping of the formwork. The connection may be tightened and untightened unlimited times and provide several further advantages if compared with other fastening methods, e.g. embedded anchor plates for welded connections. For this reason, anchor channels with channel bolts became increasingly popular and design rules were codified in the USA and Europe. However, the whole framework of the design rules is not fully consolidated to date. Therefore it is not surprising that the design rules in the USA and Europe slightly differ. Following a brief introduction of the design rule framework in the USA and Europe, the differences in the two regions are pointed out. In addition, the impact on the outcome of the design will be discussed and finally, an outlook on potential future development of the design rules for anchor channels and channel bolts is given.

1 Introduction

Anchor channels with channel bolts (aka T-bolts) allow an easy and reliable connection of components to reinforced concrete structures (Figure 1).

Figure 1: Anchor channels and channel bolts for the connection of a) catenary systems in tunnel linings and b) guiding rails and divider wall beams in elevator shafts
According to modern building design codes, concrete anchors including anchor channels with channel bolts have to be approved by independent bodies if used for safety relevant connections. Because of their high load capacity, anchor channels with channel bolts are typically used to fix heavy loads and therefore in general require qualification. Beside the careful selection of suitable and qualified products by the specifier, a professional structural design by the engineer is required for a safe connection with anchor channels and channel bolts.

The design rules for anchor channels and channel bolts are mostly based on the general design rules developed for steel fasteners anchored in concrete elements. The versatile and partly unique failure modes of anchor channel-channel bolt-systems, however, require additional and often very complex design rules. The whole framework of these design rules is not fully consolidated to date. Therefore it is not surprising that the design rules in Europe and in the USA slightly differ and consequently, the design results for identical situations show different demand-to-capacity ratios (DCR). For this reason, product specifiers and structural engineers may be confused and misguided. The goal of this paper is to provide an overview of the design of anchor channels with channel bolts and to explain the most important differences of the design rules currently to be taken into account in Europe and the USA.

2 Background of Anchor Channels

The development of anchor channels with channel bolts more than 100 years ago was driven by the need to connect transmission belts to ceiling panels made of the then upcoming reinforced concrete structures (Figure 2a). T-shaped channel bolts are locked into C-shaped channels which are fitted with anchors and cast flush in reinforced concrete elements. Conventional anchor channels with smooth channel bolts allow the transfer of tension loads (N) and shear loads perpendicular to the channel (V_y). To enable the load transfer also along the axis of the channel (V_x), serrated anchor channels and matching serrated channel bolts were developed in the 1980s, making the load transfer in all directions of the channel possible (Figure 2b). These anchor channel-channel bolt systems are very reliable since the load is transferred in all directions by mechanical interlock.

Figure 2: a) Excerpt from Jordahl’s patent for anchor channels issued 1913; b) Serrated anchor channel components and matching serrated channel bolt allow load transfer in all directions
For installation, anchor channels are attached to the formwork (Figure 3a). Anchor channels are generally furnished with filler material to prevent that concrete slurry is leaking into the profile during concreting (Figure 3b). After the concrete is set and the formwork is stripped off, the filler is removed (Figure 3c). Channel bolts inserted and twisted in the slot of the channel then allow fastening of fixtures with components at any point along its length (Figure 3d).

Figure 3: Installation sequence a) attaching of anchor channel to formwork, b) casting of concrete, c) removing of filler, d) twisting-in of channel bolt

Compared to the other steel-to-concrete fastening methods, anchor channels with channel bolts offer several benefits: Construction tolerances may be compensated by adjustable bolt fit during installation; the mechanical interlock (bolt-channel, anchor-concrete) provides a robust load transfer mechanism; no on-site welding is required as in case of anchor plates, thus no welding qualification and inspection as well as fire protection measurements required; no on-site drilling is needed as for post-installed anchors, thus no hassle with power supply, drill dust removal and accidentally cut reinforcement; installation of channel bolts in the anchor channel during construction is quick, easy and fool-proof; and connecting, disconnecting and sliding of attachment during complete life cycle of building is possible.

3 Design Rules

Research on anchor channels with channel bolts was intensified during the last two decades and included studies to determine the behavior of anchor channel-channel bolt systems when installed in thin members and with supplementary reinforcement. After the development of substantiated design equations for static loads, the influence of dynamic loads was investigated in detail. The research allowed the modification of design models originating from the general anchor technology for anchor channels with channel bolts. The developed design equations are used to verify the strengths corresponding to various possible failure modes.

Some design equations are semi-empiric and require product dependent parameters to be determined during the qualification. The most important qualification guidelines for anchor channels with channel bolts are the European Assessment Document EAD 330008-02-0601 in Europe and the Acceptance Criteria AC232 in the USA. In Europe, qualification is documented by European Technical Assessment (ETA) certificates and does not yet address seismic qualification. In the USA, Evaluation (Service) Reports (E(S)R) may include a seismic qualification for applications designed according to a more demanding Seismic Design Category (SDC).

In Europe, the design provisions for concrete anchors including anchor channels with channel bolts are provided in the pre-norm CEN/TS 1992-4 whose revision FprEN 1992-4 is currently in the
formal vote process to become EN 1992-4\textsuperscript{14}, i.e. Part 4 of the Eurocode 2, the design code for concrete structures. The amendment FprCEN/TR 17080\textsuperscript{15} was drafted to allow the design of anchor channels which channel bolts are loaded in the longitudinal direction. The latest revision of the design code for concrete structures in the USA, ACI 318\textsuperscript{16}, provides design rules for post-installed and cast-in anchors in Section 17 which were dealt within Appendix D of previous editions. The design rules for anchor channel-channel bolt systems are not yet included. For this reason, AC232\textsuperscript{8} provides amendments to the relevant clauses of ACI 318 to allow the design of anchor channels and channel bolts.

In contrast to the typically rotational symmetric post-installed and cast-in single point anchors with little more than the basic steel, concrete breakout and pullout failure modes, the geometric complex anchor channel-channel bolt-systems (Figure 4) may develop 18 different failure modes if loaded in tension and shear. The determination of the load distribution, a prerequisite for the design checks corresponding to the failure modes, further exacerbates the situation. The load distribution among the channel bolts depends on the geometry and load of the fixation where in particular the thickness of anchor plates plays a role\textsuperscript{17,18}.

Gross simplifications of the highly statically indeterminate system are made to allow an analytical estimation of the load transfer from the bolts into the anchors. A linear distribution over the influence length, $l_i = 13I_y^{0.05} \cdot s^{0.5} \geq s$ is assumed for the tension and perpendicular shear load transfer from the channel bolt to the anchor channel (Figure 5), where $I_y$ is the moment of inertia of the channel and $s$ is the spacing of the anchors\textsuperscript{3,19}. The anchor load is then calculated according to the ordinate $A'$ at the position of the anchor $i$ of a triangle with the unit height at the position of a bolt load and the base length $2l_i$. The longitudinal shear load transfer is currently simpler distributed yet more complex to be calculated because here, the anchor load depends on the edge and corner distances as well as the overall load distribution. Furthermore, different rules apply in Europe and the USA (Table 1).

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure4.png}
\caption{Schematic drawing of anchor channel with channel bolt and indicated concrete cone breakout, loaded in tension and shear: a) Cross section; b) Top view (denomination in brackets according to AC232 if deviating from EN 1992-4)}
\end{figure}
Figure 5: Schematic drawing of anchor channel with channel bolt and indicated concrete edge breakout, loaded in tension and a) longitudinal shear parallel to edge, b) transverse shear perpendicular to edge; c) load distribution (denomination in brackets according to AC232 if deviating from EN 1992-4)

Table 1: Longitudinal shear load distribution in Europe (na ≤ 3) and the USA

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>Europe: EN 1992-4, TR 17080, ETA</th>
<th>USA: ACI 318, AC232, E(S)R</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor, connection, pryout and concrete edge failure</td>
<td>Installation parallel to edge and perpendicular to edge, remote from an edge (c1 ≥ max{10hba, 60da})</td>
<td>Assigned to those (maximum) 3 anchors that result in the most unfavorable design condition</td>
</tr>
<tr>
<td>Any other failure</td>
<td>Assigned to all anchors</td>
<td>Assigned to all anchors</td>
</tr>
</tbody>
</table>

![Diagram](image1)

Figure 6: Illustration of load distribution and potential concrete breakout surfaces for channels with three anchors in Europe and the USA
Also the mostly semi-empirical design equations to calculate the resistance corresponding to the individual failure modes in tension (z-direction), perpendicular shear (y-direction) and longitudinal shear (x-direction) are partly different in Europe and in the USA. Apart from the channel loaded in bending, the design checks are carried out at the bolt (including connection to channel) and at the anchor (including connection to channel). Loads which are simultaneously acting in tension and shear, i.e. inclined loads, are taken into account by interaction equations.

In Europe, the design load equals the factored characteristic load in tension \( N_{Ed} = \gamma_F \cdot N_{Ek} \) and shear \( V_{Ed} = \gamma_F \cdot V_{Ek} \) which is opposed to the design resistance (aka capacity and strength) as the reduced characteristic resistance in tension \( N_{Rd,i} = 1/\gamma_M \cdot N_{Rk,i} \) and shear \( V_{Rd,i} = 1/\gamma_M \cdot V_{Rk,i} \) corresponding to the various failure modes. In the USA, the factored load in tension \( N_{ua} \) and shear \( V_{ua} \) is compared with the resistances in tension \( \phi N_i \) and shear \( \phi V_i \) corresponding to the various failure modes. The design safety concept of Load and Resistance Factored Design (LRFD) is in principle identical in Europe and in the USA. However, the safety factors for load and strength reduction \( \phi \) in the USA are different from the load safety factor \( \gamma_F \) and the quotient of the strength reduction divisors \( 1/\gamma_M \) in Europe.

For verification in the course of the structural design, the capacity is determined according to all design equations corresponding to the individual failure modes under the consideration of the relevant safety divisors (Europe) and factors (USA). The critical design check of the system, e.g. comprising an anchor channel with one or more channel bolts cast in concrete, is governed by the highest DCR of all possible failure modes. Since for anchor channel-channel bolt-systems several alternative load distributions have to be assumed, the critical design check cannot be determined analytically. Most important, the complexity of the design equations impedes a closed analytical solution. For these reasons, the capacity of the system has to be determined by checking every possible case.

### 4 Comparison

The analysis of the design equations in force in Europe\(^{13,15}\) and in the USA\(^{8,16}\) allow the identification of seven failure modes whose verification may lead to significant different results (Table 2). In the following, the outcomes of the corresponding design equations are compared. To separate the discussion from the concept and the level of design safety, no safety divisors and factors are taken into account. The product specific parameters were taken from the European qualification certificate ETA\(^9\) and the US counterparts, the E(S)Rs\(^{10,11}\). As a typical example, the JORDAHL® anchor channel JTA 53/34 installed parallel to the edge, with three anchors and two symmetrically arranged channel bolts JB M16 is analyzed. Cracked concrete (\( f_{ck} \approx f_{c'} = 20 \text{ MPa} \)) without supplementary reinforcement, slab thickness 200 mm, anchor spacing 250 mm, corner distance \( \infty \) and characteristic bolt spacing \( s_{lb} \) (parameter according to the CEN/TS 1992-4\(^{12}\)) were assumed. The edge distance and bolt spacing were varied for the parametric analyses. For the following discussion on the differences between the design rules in Europe and in the USA, one should bear in mind that large differences in capacities corresponding to a particular failure mode may have no effect on the overall capacity of the system if it is not the governing failure mode.
### Table 2: Verifications with significant differences in Europe and the USA

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>Europe: EN 1992-4, TR 17080, ETA</th>
<th>USA: ACI 318, AC232, E(S)R</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Steel failure channel lip</strong></td>
<td>$\mathbf{\psi}<em>{\text{BL,AL}} = \mathbf{\psi}</em>{\text{BL,AL}}^0 \cdot \psi_{\text{BL}}$</td>
<td>$\mathbf{\psi}<em>{\text{BL,AL}} = \mathbf{\psi}</em>{\text{BL,AL}}^0 \cdot \psi_{\text{BL}}$</td>
</tr>
<tr>
<td>$\mathbf{\psi}_{\text{BL,AL}}^0$ according to ETA</td>
<td>$\psi_{\text{BL}} = \min{1 / (1 + \Sigma[(1 – s_{\text{bl}}/s(N)]) \leq 1 / 0.5 \cdot h_{\text{ef}} / c }$</td>
<td>$\psi_{\text{BL}} = \min{1 / (1 + \Sigma[(1 – s_{\text{bl}}/s(N)]) \leq 1 / 0.5 \cdot h_{\text{ef}} / c }$</td>
</tr>
<tr>
<td>$\mathbf{\psi}<em>{\text{BL,AL}} = 0.5(1 + s</em>{\text{bl}}/s_{\text{LV}})$</td>
<td>$0 \ leq 1$</td>
<td>$0 \ leq 1$</td>
</tr>
<tr>
<td>$s_{\text{bl}} \geq 5d$</td>
<td>$\psi_{\text{VL}} = 0.5(1 + s_{\text{bl}}/s_{\text{LV}})$</td>
<td>$\psi_{\text{VL}} = 0.5(1 + s_{\text{bl}}/s_{\text{LV}})$</td>
</tr>
<tr>
<td>$s_{\text{LV}}$ according to ETA</td>
<td>$s_{\text{LV}}$ according to ETA</td>
<td>$s_{\text{LV}}$ according to ETA</td>
</tr>
</tbody>
</table>

An indicative value of $s_{\text{bl}} = 2b_{\text{ch}}$ may be used.

### Shear

<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>Europe: EN 1992-4, TR 17080, ETA</th>
<th>USA: ACI 318, AC232, E(S)R</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Steel failure channel lip</strong></td>
<td>$\mathbf{\psi}<em>{\text{BL,AL}} = \mathbf{\psi}</em>{\text{BL,AL}}^0 \cdot \psi_{\text{BL,AL}}$</td>
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</tr>
<tr>
<td>$\mathbf{\psi}_{\text{BL,AL}}^0$ according to ETA</td>
<td>$\psi_{\text{BL,AL}} = \min{1 / (1 + \Sigma[(1 – s_{\text{bl}}/s(N)]) \leq 1 / 0.5 \cdot h_{\text{ef}} / c }$</td>
<td>$\psi_{\text{BL,AL}} = \min{1 / (1 + \Sigma[(1 – s_{\text{bl}}/s(N)]) \leq 1 / 0.5 \cdot h_{\text{ef}} / c }$</td>
</tr>
<tr>
<td>$\psi_{\text{BL,AL}}^0 = 1$</td>
<td>$\psi_{\text{BL,AL}} = \min{1 / (1 + \Sigma[(1 – s_{\text{bl}}/s(N)]) \leq 1 / 0.5 \cdot h_{\text{ef}} / c }$</td>
<td>$\psi_{\text{BL,AL}} = \min{1 / (1 + \Sigma[(1 – s_{\text{bl}}/s(N)]) \leq 1 / 0.5 \cdot h_{\text{ef}} / c }$</td>
</tr>
<tr>
<td>$\psi_{\text{BL,AL}} = 0.5(1 + s_{\text{bl}}/s_{\text{LV}})$</td>
<td>$0 \ leq 1$</td>
<td>$0 \ leq 1$</td>
</tr>
<tr>
<td>$s_{\text{bl}} \geq 5d$</td>
<td>$\psi_{\text{VL}} = 0.5(1 + s_{\text{bl}}/s_{\text{LV}})$</td>
<td>$\psi_{\text{VL}} = 0.5(1 + s_{\text{bl}}/s_{\text{LV}})$</td>
</tr>
<tr>
<td>$s_{\text{LV}}$ according to ETA</td>
<td>$s_{\text{LV}}$ according to ETA</td>
<td>$s_{\text{LV}}$ according to ETA</td>
</tr>
</tbody>
</table>

An indicative value of $s_{\text{bl}} = 2b_{\text{ch}}$ may be used.

| Steel failure channel lip | $\mathbf{\psi}_{\text{BL,AL}} = \mathbf{\psi}_{\text{BL,AL}}^0 \cdot \psi_{\text{BL,AL}}$ | $\mathbf{\psi}_{\text{BL,AL}} = \mathbf{\psi}_{\text{BL,AL}}^0 \cdot \psi_{\text{BL,AL}}$ |
| $\mathbf{\psi}_{\text{BL,AL}}^0$ according to ETA | $\psi_{\text{BL,AL}} = \min\{1 / (1 + \Sigma[(1 – s_{\text{bl}}/s(N)]) \leq 1 / 0.5 \cdot h_{\text{ef}} / c \}$ | $\psi_{\text{BL,AL}} = \min\{1 / (1 + \Sigma[(1 – s_{\text{bl}}/s(N)]) \leq 1 / 0.5 \cdot h_{\text{ef}} / c \}$ |
| $\psi_{\text{BL,AL}} = 0.5(1 + s_{\text{bl}}/s_{\text{LV}})$ | $0 \\ leq 1$ | $0 \\ leq 1$ |
| $s_{\text{bl}} \geq 5d$ | $\psi_{\text{VL}} = 0.5(1 + s_{\text{bl}}/s_{\text{LV}})$ | $\psi_{\text{VL}} = 0.5(1 + s_{\text{bl}}/s_{\text{LV}})$ |
| $s_{\text{LV}}$ according to ETA | $s_{\text{LV}}$ according to ETA | $s_{\text{LV}}$ according to ETA |

An indicative value of $s_{\text{bl}} = 2b_{\text{ch}}$ may be used.
<table>
<thead>
<tr>
<th>Failure Mode</th>
<th>Shear (cont.)</th>
<th>Tension or shear</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete edge failure, longitudinal loading, installation perpendicular to edge</td>
<td>[ V_{R,k,e} = V_{R,k} = A_{s,y} \cdot \psi_{s,y} \cdot \psi_{y,e} \cdot \psi_{s,e} \cdot \psi_{y,e} \cdot \psi_{s,e} ]</td>
<td></td>
</tr>
<tr>
<td></td>
<td>[ V_{R,k} = k_0 \cdot \alpha \cdot \beta \cdot (h_{ef} - h_a)^{0.5} \cdot f_k^{0.5} \cdot c_1^{1.5} ]</td>
<td>[ N_{u,ae} \leq \phi N_{u,a} ]</td>
</tr>
<tr>
<td></td>
<td>[ k_0 = 1.7 \text{ for cracked concrete} ]</td>
<td>[ N_{u,ae} \text{ according to strut-and-tie models where shear loads are multiplied by } (1 + e_i/z), ]</td>
</tr>
<tr>
<td></td>
<td>[ \alpha = 0.1(h_{def}/c_1)^{0.5} ]</td>
<td>[ e_i \text{ is the distance between reinforcement and force,} ]</td>
</tr>
<tr>
<td></td>
<td>[ \beta = 0.1(d/c_1)^{0.2} ]</td>
<td>[ z = 0.85 \cdot (h - h_a - \phi) \leq \min{2h_{ef}; 2c_1} ]</td>
</tr>
<tr>
<td></td>
<td>[ A_{e,V} = (1.5c_1 + \min{c_2; 1.5c_1}) \cdot \min{h; 1.5c_1} ]</td>
<td>[ z = 0.85 \cdot (h - h_a - d_b) \leq \min{2h_{ef}; 2c_1} ]</td>
</tr>
<tr>
<td></td>
<td>[ \psi_{s,V} = 0.7 + 0.3c_2/1.5c_1 \leq 1 ]</td>
<td>[ \text{is the internal lever arm of reinforced concrete element} ]</td>
</tr>
<tr>
<td></td>
<td>[ \psi_{h,V} = (1.5c_1/h)^{0.5} \geq 1 ]</td>
<td>[ \zeta = \min{2h_{ef}; 2c_1} \leq \min{2h_{ef}; 2c_1} ]</td>
</tr>
<tr>
<td></td>
<td>[ \psi_{s,V} = 1.0 \text{ for cracked concrete} ]</td>
<td>[ \zeta = \min{2h_{ef}; 2c_1} \leq \min{2h_{ef}; 2c_1} ]</td>
</tr>
<tr>
<td></td>
<td>[ 1.4 \text{ if longitudinal and perpendicular edge reinforcement or for uncracked concrete} ]</td>
<td>[ N_{u,ae} \leq \phi N_{u,a} ]</td>
</tr>
<tr>
<td>Concrete edge failure, longitudinal loading, installation parallel to edge</td>
<td>[ V_{R,k,e} = 2V_{R,k} = A_{s,y} \cdot \psi_{s,y} \cdot \psi_{y,e} \cdot \psi_{s,e} ]</td>
<td>[ N_{u,ae} \leq \phi N_{u,a} ]</td>
</tr>
<tr>
<td></td>
<td>[ V_{R,k} = A_{s,y} \cdot (1.5c_1 + \sum s_i + \min{c_2; 1.5c_1}) \cdot \min{h; 1.5c_1} ]</td>
<td>[ N_{u,ae} \text{ according to strut-and-tie models where shear loads are multiplied by } (1 + e_i/z), ]</td>
</tr>
<tr>
<td></td>
<td>[ \sum s_i \text{ summarized anchor spacing} ]</td>
<td>[ e_i \text{ is the distance between reinforcement and force,} ]</td>
</tr>
<tr>
<td></td>
<td>[ A_{e,V} = (1.5c_1; \min{c_2; 1.5c_1}) ]</td>
<td>[ z = 0.85 \cdot (h - h_a - d_b) \leq \min{2h_{ef}; 2c_1} ]</td>
</tr>
<tr>
<td></td>
<td>[ \psi_{s,V} = 1.0 \text{ as above} ]</td>
<td>[ \zeta = \min{2h_{ef}; 2c_1} \leq \min{2h_{ef}; 2c_1} ]</td>
</tr>
<tr>
<td></td>
<td>[ \psi_{h,V} = 1.0 \text{ as above} ]</td>
<td>[ \zeta = \min{2h_{ef}; 2c_1} \leq \min{2h_{ef}; 2c_1} ]</td>
</tr>
<tr>
<td></td>
<td>[ n_s \text{ number of anchors} ]</td>
<td>[ N_{u,ae} \leq \phi N_{u,a} ]</td>
</tr>
</tbody>
</table>

1. For anchor channels \( c_{ch} = s_{ch}/2 = 1.5h_a \) according to EAD (EAD 330008-02-0601)
2. For fasteners including anchor channels in cracked concrete \( k_5 = 8.7 \) and in uncracked concrete \( k_5 = 12.2 \)
4.1 Tension

Steel failure channel lip: The equation to take the influence of neighboring bolts into account was recently changed in the USA to find a more reasonable approach and to cover cases where neighboring bolts are situated at both sides. The example shows that the difference of the values derived by the equations stipulated in the USA and in Europe are rather small (Figure 7a). However, the approach should be mirrored to Europe to cover also the designs with more than two neighboring bolts and to harmonize the design rules in both regions.

Concrete splitting failure: In Europe, a separate critical edge distance $c_{cr,sp}$ and anchor spacing $s_{cr,sp}$ for splitting is introduced in a separate verification for concrete splitting failure. This is not the case in the USA, where this check is included in the check for concrete cone failure. Furthermore, the corresponding reduction factors $\psi_{h,sp}$ and $\psi_{cp,N}$ in Europe and the USA, respectively, have a different formulation. In the presented example, the difference accumulates to around 20% in total (Figure 7b). Since splitting is here typically a secondary phenomenon, the difference between the two approaches will be seldom relevant. Harmonization, however, would be appreciated.

Concrete blow-out failure: Due to the load distribution effect, a row of anchors have a larger concrete blow-out capacity if compared with a single anchor. In the USA, this group effect is expressed by the increase factor $\psi_{g,Nb}$. In Europe, it was decided not to taken into account the effect for simplicity. In addition, the factor for the calculation of basic concrete blow-out strength is in Europe significantly lower if compared with the USA as the example demonstrates (Figure 7c). Since the approach of the concrete blow-out failure verification is currently under discussion among the experts, the approach may be modified in Europe and subsequently in the USA which could mitigate the differences between the two codes.

![Graphs illustrating the most significant differences between the verification in Europe and in the USA, failures in tension](image)

4.2 Shear

Steel failure channel lip: In Europe, the influence of neighboring bolts is not only taken into account for tension but also for shear. Currently, the influence in shear is neglected in the USA and for this reason, the calculated capacities differ significantly as shown by the example (Figure 8a). It would be meaningful to add a factor in the USA similar to the factor to take into account the effect of
neighboring anchors in tension. The same equation should be then mirrored to Europe in order to harmonize, to find a more reasonable approach and to cover cases where neighboring bolts are situated at both sides.

Concrete edge failure, longitudinal loading, installation perpendicular to edge: Though the verification for concrete edge failure of longitudinally loaded anchor channels installed perpendicular to the edge is based on the same approach in Europe and the USA, the equations slightly differ. The core part of the equation used in Europe was adopted from the concrete anchor technology and is valid for channels with up to three anchors only. A simpler and more conservative approach for the core part of the equation is used in the USA. In Europe, however, the assumption that the anchor closest to the edge takes the complete load is more conservative than the assumption made in the USA (Figure 6). Here, the number anchors \( n_a \) is not restricted and the load is assigned to the three front anchors closest to the edge. Experience is required to realize that it is advisable to carry out the design check under the case-wise assumptions that the breakout is initiated at the anchor closest to the edge or at any anchor further to the back, where anchors situated in the concrete breakout body do not carry any load. Furthermore, the breakout surfaces originating at several anchors may merge if the anchor spacing is smaller than the edge distance (\( s < c_1 [c_{a1}] \)) where a more conservative assumption would be that the anchor nearest to the edge has to take the full load. Since the design check in the USA may refer to a group of anchors, the capacities shown in the diagram for the example are the fractions of one activated anchor to allow a better comparability with the considerably lower capacity according to the approach in Europe (Figure 8b).

Concrete edge failure, longitudinal loading, installation parallel to edge: In Europe, also longitudinally loaded anchor channels installed parallel to the edge are regulated for up to three equally loaded anchors only whereas the number anchors \( n_a \) is not limited in the USA (Figure 6). Here, the load is assigned to those three anchors that result in the most unfavorable design condition. For this reason, iterations are required in the USA to identify the most critical interaction design check as a function of the distribution of perpendicular shear and tension loading. Note that in Europe the capacity is determined for the concrete breakout of all three anchors and then averaged for one anchor whose capacity is distinctly lower for the analyzed example (Figure 8c).

**Figure 8:** Examples illustrating the most significant differences between the verification in Europe and in the USA, failures in shear

![Diagram](image-url)
4.3 Tension or shear

Steel and anchorage failure supplementary [anchor] reinforcement: The capacity of anchor channel-channel bolt-systems may be increased by providing hairpin reinforcement (for shear) or stirrups (for tension) which is referred to as supplementary reinforcement in Europe and anchor reinforcement in the USA. The verification of steel and anchorage failure of this reinforcement is carried out according to the reinforced concrete design codes in Europe, EN 1992-1-1\textsuperscript{20}, and in the USA, ACI 318\textsuperscript{16}. These codes have identical approaches for the design of the steel capacity whereas the anchorage capacity is designed differently. Furthermore, supplementary reinforcement of anchor channels in Europe may be designed with anchorage lengths smaller than the development length, whereas the full development length is required in the USA for all SDCs. These differences inevitably lead to unequal design results. Identical design outcomes in Europe and the USA would require the harmonization of EN 1992 and ACI 318 with regard to the provisions for the design of anchorages.

5 Summary

This paper briefly explains the background of anchor channels and channel bolts used to connect steel fixings of components to reinforced concrete structures. The complex load bearing behavior characterized by many different failure modes is briefly introduced. An overview of the critical design rules for anchor channel-channel bolt systems which significantly differ between Europe and the USA is provided to allow their comparison and discussion.

The largest difference in the formulation and outcome has been identified at the design rules for anchor channels with channel bolts under longitudinal loading. The design rules should be amended in Europe for channels with more than three anchors. In the USA, clarification is needed in particular for anchor channels installed perpendicular and parallel to the edge with corner effects with regard to the assumed concrete breakout surface for the calculation of the capacity. Research is advisable to find a simple and economic approach how to design anchor channels with channel bolts subjected to longitudinal loading.

The design equations in Europe and the USA should be generally harmonized to streamline the codes. Different design outcomes for identical anchor channel-channel bolt-systems cannot be justified and confuse the designer. The design code in Europe should be urgently amended for seismic applications of anchor channel-channel bolt-systems. In the USA, all design rules for anchor channel-channel bolt-systems should be included in the design code for structural concrete.

The views expressed in this paper are the views of the authors only and do not necessarily reflect the views of Jordahl GmbH and the University of Stuttgart.

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CONCRETE BREAKOUT STRENGTH OF ANCHOR CHANNELS IN EDGE AND CORNER SITUATIONS UNDER PERPENDICULAR SHEAR LOADING

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ABSTRACT

Serrated anchor channels located close to the edge and subjected to shear loads catch the attention of many researchers1,2. Especially, the influence of corners on the concrete breakout strength has seldom been quantified in physical experiments or documented in literature. This study is motivated by deviations between recent design guides3,4, which both provide conservative estimates on the concrete breakout strength, and the attained test results. In addition to standard tests on close to edge situations, a refined experimental setup for corner situations exposed to perpendicular shear loads was developed. Test series on four types of serrated cast in concrete anchor channels were carried out varying slab thicknesses as well as edge and corner distances. The concrete test members were fabricated in three different configurations; without reinforcement, with minimal surface reinforcement according to EN 19925, or with additional stirrups located nearby the anchor channels. An increase of the failure load up to 48 % has been achieved employing surface reinforcement. Also additional stirrups led to a further increase up to 58 %. Interestingly, this behavior cannot be expected from the current design guides3,4, wherein the concrete breakout strength of reinforced concrete members is noticeably underestimated. Moreover, it has been demonstrated that with adequate structural design and following established detailing rules for supplementary reinforcement, corner situations do not influence the concrete breakout strength.

1 Introduction

Anchor channels cast in plain concrete that are located close to the edge and subjected to perpendicular shear loads towards the edge may fail in concrete edge failure6. The associated shear failure load may be predicted according to EN 1992-43. Test results suggest that supplementary reinforcement might increase the failure load6. With supplementary reinforcement a specimen may show steel failure or anchorage failure. In general the minimum of both is decisive and determines the shear resistance. But which one turns out in testing depends on various factors. According to EN 1992-4, concrete edge failure must not be verified at all when providing supplementary reinforcement. CEN/TR 170807 defines an improved model for designs with supplementary reinforcement, which provides a more detailed concept to prove reinforcement in form of stirrups.
Preliminary tests in our lab (IDs A1-A3 in Table 2) delivered a significant increase of the failure load when supplementary reinforcement is contained in specimens. Indeed, such an increase is not predicted by recent design guides\textsuperscript{3,7}. Therefore, a special test program has been set up to investigate the actual behavior of anchor channels.

In this contribution the experimental results on four types of serrated anchor channels embedded in concrete slabs are presented. The slabs possess a variable thickness that increases with the channel size. Moreover, the amount of reinforcement is varied. Additional to standard tests in edge situations all profile types were tested in corner situations, too. Therefore, a refined test setup has been developed. Testing aims at determining the impact of reinforcement and corners on the concrete breakout strength. Up to now the influence of corners has seldom been quantified in physical experiments or documented in literature at all. Thus, the results are compared to estimates according to recent design guides. Emphasis is set on the deviations. Based on the outcome a suggestion is made how the design guides might be enhanced to provide more realistic estimates.

## 2 Test Program

Experimental investigations have been conducted with serrated anchor channels of four different types. The hot-rolled profiles varied in heights and widths as summarized in Table 1. All channels were provided with two round anchors riveted to the back with varying anchor and end distances which give individual total lengths.

<table>
<thead>
<tr>
<th>Profile type</th>
<th>Width [mm]</th>
<th>Height [mm]</th>
<th>Length [mm]</th>
<th>Anchor distance s [mm]</th>
<th>End distance [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>29/20</td>
<td>29</td>
<td>20</td>
<td>250</td>
<td>200</td>
<td>25</td>
</tr>
<tr>
<td>38/23</td>
<td>38</td>
<td>23</td>
<td>300</td>
<td>250</td>
<td>25</td>
</tr>
<tr>
<td>53/34</td>
<td>53</td>
<td>34</td>
<td>320</td>
<td>250</td>
<td>35</td>
</tr>
<tr>
<td>64/44</td>
<td>64</td>
<td>44</td>
<td>320</td>
<td>250</td>
<td>35</td>
</tr>
</tbody>
</table>

All anchor channels were subjected to a perpendicular shear load by mounting two bolts in the channel directly above the anchors. The experiments were conducted in our lab displacement-controlled with a constant velocity until the maximum load was reached. Since the maximum load is not known in advance, the velocity can be increased to fasten the procedure once a drop of the load is observed during testing. Meanwhile the displacement of the anchor channel was monitored employing transducers on the back of the load transmission plates. In general, the concrete slabs were manufactured in alternative configurations.

For reference the profile type 53/34 serves. In an edge situation (cf. Figure 1, left) the channel was attached to plain concrete slabs, slabs with minimal surface reinforcement according to EN 1992\textsuperscript{5} and slabs having additional stirrups close to the anchor channel (IDs A1-A3, Table 2). While A2 has no additional stirrups, A3 varies the diameter of the stirrups, too. All three configurations were tested several times to allow for statistics. Details are provided in Section 3. In general, these tests aimed at assessing an influence of different amounts and layouts of reinforcement.
Additionally, all types of profiles were embedded in reinforced concrete slabs with minimal surface reinforcement and additional stirrups (cf. Table 2). Consistently one stirrup was arranged at the channel’s center between the two anchors while the other two stirrups were placed with an offset of 200 mm to both sides. The diameter of the stirrups varied with increasing channel size while the minimum surface reinforcement was kept constant. In all configurations bar reinforcement along the edge was provided locally according to an appropriate truss model (strut and tie model).

### Table 2: Total test program

<table>
<thead>
<tr>
<th>ID</th>
<th>Profile type</th>
<th>Slab thickness h [mm]</th>
<th>Edge distance c₁ [mm]</th>
<th>Corner distance c₂ [mm]</th>
<th>Reinforcement</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>53/34</td>
<td>250</td>
<td>200</td>
<td>-</td>
<td>Ø6/200</td>
<td>-</td>
</tr>
<tr>
<td>A2</td>
<td>53/34</td>
<td>250</td>
<td>200</td>
<td>-</td>
<td>Ø6/200</td>
<td>-</td>
</tr>
<tr>
<td>A3</td>
<td>53/34</td>
<td>250</td>
<td>200</td>
<td>-</td>
<td>Ø6/200 3 Ø6, s = 150 2 Ø10</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Ø6/200 3 Ø8, s = 150 2 Ø10</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>29/20</td>
<td>120</td>
<td>100</td>
<td>-</td>
<td>Ø6/200 3 Ø6, s = 200 2 Ø10</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>38/23</td>
<td>150</td>
<td>150</td>
<td>-</td>
<td>Ø6/200 3 Ø8, s = 200 2 Ø10</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>53/34</td>
<td>200</td>
<td>200</td>
<td>-</td>
<td>Ø6/200 3 Ø8, s = 200 2 Ø10</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>64/44</td>
<td>250</td>
<td>250</td>
<td>-</td>
<td>Ø6/200 3 Ø10, s = 200 2 Ø10</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>29/20</td>
<td>120</td>
<td>100</td>
<td>105</td>
<td>Ø6/200 3 Ø6, s = 200 2 Ø10</td>
<td></td>
</tr>
<tr>
<td>H</td>
<td>38/23</td>
<td>150</td>
<td>150</td>
<td>155</td>
<td>Ø6/200 3 Ø8, s = 200 2 Ø10</td>
<td></td>
</tr>
<tr>
<td>J</td>
<td>53/34</td>
<td>200</td>
<td>200</td>
<td>200</td>
<td>Ø6/200 3 Ø8, s = 200 2 Ø10</td>
<td></td>
</tr>
<tr>
<td>K</td>
<td>64/44</td>
<td>250</td>
<td>250</td>
<td>250</td>
<td>Ø6/200 3 Ø10, s = 200 2 Ø10</td>
<td></td>
</tr>
</tbody>
</table>

Besides standard testing in edge situations a refined setup for corner situations has been developed (Figure 1, right). Due to the anchor channel located at a slab’s corner, just one horizontal support (A) has been arranged in line with the shear force. In contrast to the edge situation (Figure 1, left), where the shear force is born by two symmetrically arranged horizontal supports, a turning moment results from the eccentricity between the shear force and the support. To prevent from rotation and to take over the forces, two supports (B, C) have been placed laterally at the opposite sides of the slab. The clear distance between the anchor and the support(s) at load application is at least $2.5 \cdot c₁$ according to the European Assessment Document for anchor channels (EAD⁸). Vertical supports to prevent the slab from uplifting have been arranged on the back. However, these are not shown in Figure 1 to keep the overview.
The full test program with all data concerning individual edge and corner distances and the belonging reinforcement is summarized in Table 2. Apart from the reference members in edge situation (A1-A3) all four profiles have been tested close to an edge (IDs C-F) and at a corner (IDs G-K). Please note: When testing edge situations the corner distances were chosen sufficiently large to prevent from influencing the results.

3 Results of the Experimental Tests

Next, the experimental results are analyzed. The maximum loads obtained from testing are contrasted to investigate the single modifications. That way, the impact of reinforcement is quantified and the influence of corners in reinforced concrete members is enlightened. Besides the maximum loads at failure, the load displacement plots help to discuss the different bearing behavior. In most cases, steel failure of the reinforcement occurred after first cracks announced forthcoming concrete breakout. However, some tests have been terminated after peak load but before steel failure explicitly occurred. Anchorage failure has not been observed at all.

3.1 Influence of Reinforcement

The influence of reinforcement may be analyzed best comparing the reference tests A1, A2 and A3 one to another. Figure 2 shows the mean failure loads obtained from the tests. Obviously, the failure load increases with higher amounts of reinforcement. Tests in plain concrete (A1) achieve the smallest failure load with a mean value of 81 kN derived from three tests. Employing surface reinforcement in member A2 (Ø6 / 200 mm) leads to an increase of 48 %. The mean value of 120 kN is derived from four tests. Additional reinforcement in form of stirrups causes a further increase of 31 % (stirrups: Ø 6 mm) and 58 % (stirrups: Ø 8 mm). Both configurations with three stirrups are repeated twice. The mean failure loads are 157 kN (Ø 6 mm) and 189 kN (Ø 8 mm).

![Figure 2: Mean failure loads obtained from reference tests A1, A2, A3](image)

The influence of the reinforcement can also be exemplified by the selected load displacement plots shown in Figure 3. When the maximum is reached, the load drops significantly for the plain concrete member A1. By contrast, in case of the members A2 and A3 the load is transferred to the reinforcement. The corresponding curves exhibit a constant but lower load plateau when passing the failure load. Here, a more ductile failure due to the supplementary reinforcement occurs.
3.2 Influence of Corners

The impact of corners on the breakout strength of anchor channels in reinforced concrete members may be analyzed comparing the results of configurations C, D, E and F (edge situation) to G, H, J, and K (corner situation). The anchor channels in both situations have been placed at the same edge distances $c_1$ regarding the profile type. In the corner situation, displayed in Figure 4, a second measure, named $c_2$, defines the distance of the closer anchor to the corner. Again these corner distances depend on the corresponding profile type while the slab thicknesses and reinforcement amounts are constant for each configuration according to Table 2. To provide a sufficient anchoring length regarding the local truss model (strut and tie model) the reinforcement along the edge is bent around the corner.

The test results are contained in Table 3. Again mean failure loads have been computed from individual numbers of tests. As expected the bigger the profile the higher the breakout strength of concrete is regardless the situation. Comparing edge and corner situations very similar bearing
capacities are obtained. This is also supported by the ratio of corner to edge results depicted in Figure 5 which is always close to 1. The highest capacity loss, actually 7 %, can be stated for profile type 38/23, while the highest gains are obtained for the largest profile (type 64/44). In conclusion one cannot state a significant impact of corner situations on concrete’s breakout strength for equivalently reinforced members.

Table 3: Test results of anchor channels located in edge and corner situations

<table>
<thead>
<tr>
<th>Profile type</th>
<th>Mean failure load</th>
<th>Ratio Corner / Edge</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Edge situation</td>
<td>Corner situation</td>
</tr>
<tr>
<td></td>
<td>No. of tests [kN]</td>
<td>No. of tests [kN]</td>
</tr>
<tr>
<td>29/20</td>
<td>7 47.0</td>
<td>4 46.6</td>
</tr>
<tr>
<td>38/23</td>
<td>8 118.8</td>
<td>4 110.6</td>
</tr>
<tr>
<td>53/34</td>
<td>6 168.2</td>
<td>4 166.3</td>
</tr>
<tr>
<td>64/44</td>
<td>7 219.3</td>
<td>4 230.8</td>
</tr>
</tbody>
</table>

4 Comparison to Design Guidelines

Next, the failure loads obtained from the experiments are compared to estimates regarding recent design guidelines\(^3,7\). An analysis is performed for all profile types embedded in reinforced concrete in edge situations (IDs C-F) and for profile type 53/34 in plain concrete (A1) as well as with minimal surface reinforcement (A2) as references. In general, three potential failure mechanisms must be considered when calculating an anchor channel’s resistance (cf. section 1):

a) concrete edge failure
b) steel failure of the reinforcement
c) anchorage failure of the reinforcement

To determine the characteristic resistance \(V_{Rk,c}\) for concrete edge failure, EN 1992-4 provides a multiplicative approach according to Equation (1):

\[
V_{Rk,c} = V_{Rk,c}^0 \cdot \psi_{ch,s,v} \cdot \psi_{ch,c,v} \cdot \psi_{ch,h,v} \cdot \psi_{ch,90,v} \cdot \psi_{re,v}
\]  

\(V_{Rk,c}^0\) is multiplied with several factors to account for the concrete edge resistance of adjacent anchors (\(\psi_{ch,s,v}\)), corners (\(\psi_{ch,c,v}\)), the member’s thickness (\(\psi_{ch,h,v}\)), shear loads acting parallel to the edge (\(\psi_{ch,90,v}\)) and reinforcement located on the edge (\(\psi_{re,v}\)).

\[
V_{Rk,c}^0 = k_{12} \cdot \sqrt{f_{ck}} \cdot c_1^{4/3}
\]  

Basis is the characteristic resistance \(V_{Rk,c}^0\) of a single anchor loaded perpendicular to the edge as a function of concrete’s compressive strength and dependent on the edge distance \(c_1\) as well as a channel specific factor \(k_{12}\).

By contrast, the resistances in case of steel and anchorage failure can be calculated according to EN 1992-4 or CEN/TR 17080 whereas the latter one offers more detailed methods. Regarding EN 1992-4 the characteristic resistance for steel failure \(N_{Rk,re}\) follows from Equation (3),

\[
N_{Rk,re} = k_{10} \cdot \sum_{i=1}^{n_{re}} A_{s,ri} \cdot f_{yk,re}
\]
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wherein $k_{10}$ denotes the efficiency of reinforcement, $n_{re}$ the number of effective bars per anchor, $A_{s, re, i}$ a rebar’s cross-sectional area and $f_{yk, re}$ the characteristic yield strength of the reinforcement. Moreover, the design resistance in case of anchorage failure maybe computed according to Equation (4) as the sum of contributions of all individual bars involved.

$$N_{Rd,a} = \sum_{i=1}^{n_{re}} N_{Rd,a}^{0}$$

$$N_{Rd,a}^{0} = \frac{l_{1} \cdot \pi \cdot \phi \cdot f_{b,d}}{\alpha_{1} \cdot \alpha_{2}} \leq A_{s, re} \cdot f_{yk, re} \cdot \frac{1}{\gamma_{Ms, re}}$$

The design resistance $N_{Rd,a}^{0}$ of a single rebar in Equation (5) depends on its anchorage length $l_{1}$, its diameter $\phi$ and the design bond strength $f_{b,d}$ as well as two factors $\alpha_{1}$ and $\alpha_{2}$, that consider the form of the rebar and the minimum concrete cover according to EN 1992-1-1\textsuperscript{5}. It is always limited by the axial force of a single rebar, in which $\gamma_{Ms, re}$ denotes the partial factor of steel.

Subsidiary to EN 1992-4 CEN/TR 17080 provides a refined so called improved model to compute the characteristic tensile resistance of a stirrup according to Equation (6).

$$N_{Rk,a,i} = \psi_{cr} \cdot \left( N_{Rk, hook, i} + N_{Rk, bond, i} \right) \leq A_{s, i} \cdot f_{yk}$$

$$N_{Rk, hook, i} = \psi_{1} \cdot \psi_{2} \cdot \psi_{3} \cdot A_{s} \cdot f_{yk} \cdot \left( \frac{f_{ck}}{25} \right)^{0.1}$$

$$N_{Rk, bond, i} = \pi \cdot \phi \cdot l_{1,i} \cdot f_{bk}$$

While $N_{Rk, hook, i}$ denotes the characteristic tensile resistance of a rebar’s hooked end employing three efficiency factors $\psi_{1}$, $\psi_{2}$ and $\psi_{3}$ of the supplementary reinforcement, $N_{Rk, bond, i}$ covers the bond’s contribution along the anchorage length $l_{1}$. These two summands are multiplied by $\psi_{cr}$ to account for potential cracks along the longitudinal axis of stirrups. Again the characteristic tensile resistance $N_{Rk}$ is limited by the axial force in the rebar. At last, it is converted to a shear resistance $V_{Rk}$ of an anchor channel with respect to the inner lever arm $z$ and the distance between load application and reinforcement $e_s$ according to Equation (9) and Figure 6.

$$V_{Rk} = \frac{N_{Rk}}{(\frac{e_s}{z} + 1)}$$

Figure 6: Surface reinforcement takes the shear forces – forces in the reinforcement [EN 1992-4, Figure 6.8 b)]

The experimental and computational results, both as characteristic values, are listed in Table 4 and displayed in Figure 7. Therein, the experimental failure loads represent 5 % fractiles computed from the test results and assuming a Gaussian distribution. To calculate the estimates according to the design guidelines concrete’s mean compressive strength, measured in associated tests, has been used.
Similarly the yield strength of steel is employed to calculate the resistance associated with steel failure of the reinforcement. Since no channel specific factors $k_{12}$ are available yet, the minimum values according to EN 1992-4 are assumed ($k_{12} = k_{ucr,V} = 6.3$ for non-cracked concrete). It must be noted, that the results in Table 4 represent the resistance of a total channel with two anchors.

Table 4: Characteristic failure loads $F_{R,k}$ [kN] obtained from tests and design guidelines

<table>
<thead>
<tr>
<th>ID</th>
<th>Profile type</th>
<th>Experiment</th>
<th>EN 1992-4 Concrete edge failure</th>
<th>Reinforcement</th>
<th>EN 1992-4 Reinforcement</th>
<th>CEN/TR 17080 Concrete edge failure</th>
<th>CEN/TR 17080 Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>53/34</td>
<td>68.6</td>
<td>32.3</td>
<td>*</td>
<td>*</td>
<td>*</td>
<td>*</td>
</tr>
<tr>
<td>A2</td>
<td>53/34</td>
<td>104.8</td>
<td>32.3</td>
<td>23.8</td>
<td>23.8</td>
<td>**</td>
<td>**</td>
</tr>
<tr>
<td>C</td>
<td>29/20</td>
<td>44.1</td>
<td>14.4</td>
<td>31.2</td>
<td>16.2</td>
<td>31.2</td>
<td>21.6</td>
</tr>
<tr>
<td>D</td>
<td>38/23</td>
<td>107.8</td>
<td>22.2</td>
<td>79.1</td>
<td>28.5</td>
<td>57.6</td>
<td>56.4</td>
</tr>
<tr>
<td>E</td>
<td>53/34</td>
<td>148.0</td>
<td>30.0</td>
<td>81.2</td>
<td>56.0</td>
<td>59.1</td>
<td>59.1</td>
</tr>
<tr>
<td>F</td>
<td>64/44</td>
<td>179.3</td>
<td>38.1</td>
<td>138.7</td>
<td>84.1</td>
<td>93.7</td>
<td>93.7</td>
</tr>
</tbody>
</table>

* plain concrete
** straight bars are not assumed to be effective

![Figure 7: Comparison of characteristic failure loads $F_{R,k}$ obtained from experiments and according to the guidelines EN 1992-4 and CEN/TR 17080](image)

Figure 7 displays the results of the edge tests. At a first glance one realizes that both guidelines, EN 1992-4 and CEN/TR 17080, provide conservative estimates compared to the experimental results in general. In some cases the deviations might be even large. While in case of plain concrete (A1)
only edge failure can be contrasted to the experimental results, slab A2 contains surface reinforcement. Hence, steel and anchorage resistances are obtained from EN 1992-4. Since no stirrups are provided no new insights are gained from the improved model of CEN/TR 17080.

Comparing the computational results of A1 and A2 it can be confirmed that the steel and anchorage resistances might be smaller than the concrete edge one, which is here of equivalent size due to equal configuration (cf. Table 2). Besides, in testing the failure load of A2 was actually found 48% higher than the one of A1 (cf. Figure 2). Apart from this single exception concrete edge failure is generally associated with the lowest load level of all potential failure modes.

All other slabs (C-F) contain stirrups and thus are suited to oppose the guidelines’ predictions. Steel failure according to EN 1992-4 is always predicted greater or equal to CEN/TR 17080 and vice versa regarding anchorage failure. However, with both guidelines, the estimates of anchorage failure are limited by steel failure. This means, if the concrete failure cone (breakout body) cannot be tied back by reinforcement, a specimen fails. This limit is active in case of the slabs A2, E and F. Otherwise anchorage failure could be shifted towards steel failure at maximum improving design details by means of the anchoring length or the concrete strength class.

Deviations between the two design guidelines are traced back on the different assumptions made for effective reinforcement. In CEN/TR 17080 only stirrups are considered to be effective, while EN 1992-4 accounts for straight bars (surface reinforcement), too. This generally leads to higher estimates in case of steel failure according to EN 1992-4. On the other hand higher estimates in case of anchorage failure are predicted by CEN/TR 17080. Although only three stirrups are taken into account CEN/TR 17080 leads to higher resistances compared to EN 1992-4, where up to four straight bars when crossing the concrete’s failure cone (breakout body) are considered effective.

Also the predicted failure modes do not comply well with the experiments where steel failure occurred mostly and anchorage failure has not been observed at all (cf. Figure 8).

![Steel failure of the reinforcement (ID A3)](image)

Figure 8: Steel failure of the reinforcement (ID A3)

Since all estimates are quite conservative, a summation of two resistances in an additive approach is considered next, which is totally different to the guidelines. Therein, a contribution of concrete to the load transfer is generally disregarded, when supplementary reinforcement is implemented. Neglecting this fact Figure 9 and 10 show accumulated resistances of concrete and reinforcement
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(both mechanisms according to EN 1992-4 and CEN/TR 17080), separately. They indicate that considering the contribution of concrete leads to higher estimates which agree much better with the experimental test results and do not exceed it. Since anchorage failure in the concrete cone (breakout body) is decisive for most configurations the graphs at top of Figure 9 and 10 mark the upper limit that could be expected.

![Figure 9: Addition of characteristic failure loads $F_{R,k}$ obtained from EN 1992-4](image)

![Figure 10: Addition of characteristic failure loads $F_{R,k}$ obtained from CEN/TR 17080](image)

Alternatively, the failure loads may also be calculated according to AC232. In case of concrete edge failure ($V_{cb,y}$) the equations (10) and (11) do mainly differ from EN 1992-4 by the channel factor $\alpha_{ch,v}$ (cf. equations (1) and (2), channel factor: $k_{12}$).

$$V_{cb,y} = V_b \cdot \psi_{s,v} \cdot \psi_{co,v} \cdot \psi_{c,v} \cdot \psi_{h,v}$$ (10)

Therein $V_b$ is multiplied with several factors to account for the influence of adjacent anchors ($\psi_{s,v}$), corners ($\psi_{co,v}$), cracking of concrete at service loads ($\psi_{c,v}$) and the thickness of the member ($\psi_{h,v}$).

$$V_b = \lambda \cdot \alpha_{ch,v} \cdot \sqrt{f'_c} \cdot c_{a1}^{4/3}$$ (11)
$V_b$, the basic concrete breakout strength in shear perpendicular to the channel of a single anchor, is a function of the concrete’s compressive strength and also depends on the edge distance $c_{a1}$, a channel specific factor $\alpha_{ch,v}$ and a modification factor for lightweight concrete ($\lambda$).

In comparison to EN 1992-4 the estimates calculated in case of concrete edge failure according to AC232 are low, since the minimum channel factor $\alpha_{ch,v} (= 4.0)$ is always lower than $k_{12} (= k_{cr,V} = 4.5$ for cracked concrete) defined in EN 1992-4. Additionally, the reinforcement in all configurations must be considered not effective according to maximum distances between stirrups (152 mm = 6 in). Thus the estimates according to AC232 have turned out being even more conservative, they are not considered further on.

5 Conclusion and Remarks

In this paper, the experimental results on serrated cast in concrete anchor channels are introduced and compared to recent guidelines$^{3,4,7}$. The focus lies on the breakout strength of concrete. The test series comprises four types of anchor channels attached to plain and reinforced concrete slabs. Thereby, distinction is made between surface reinforcement in form of straight bars and additional stirrups. Two arrangements of the anchor channels have been considered, too; close to a slab’s edge and at corner.

By comparison of edge and corner situations it turned out that the breakout strength of concrete is not reduced in corners if equivalent reinforcement is provided. But, to ensure an appropriate truss model (strut and tie model) the edge reinforcement has to be bend around the corner to provide a sufficient anchoring length. Unfortunately, these findings cannot be mapped one to one on arbitrary configurations. Hence, further research is needed to assess configurations e.g. with smaller corner distances. In these stirrups will probably get an offset and affect the truss model (position of strut and tie) and thus influence the breakout strength.

Furthermore, the influence of supplementary reinforcement on the breakout strength was analyzed and compared to estimates from recent design guides$^{3,4,7}$. From these tests it follows, that additional reinforcement leads to significantly higher failure loads meanwhile recent design guides deliver conservative estimates for the failure load only and do not correspond well to the observations in the experiments concerning both, failure loads and modes.

Even if the estimates for concrete edge resistance and resistance of the reinforcement are accumulated, the results stay below the experimental ones. However, neglecting the contribution of concrete to the load transfer in reinforced configurations does not comply with the actual behavior observed in the experiments at all. Since so far all tests were conducted with non-cracked concrete further tests in cracked conditions are recommended. In cracked concrete reduced failure loads are expected in tests as well as in calculations according to the design guidelines. It is questionable if an accumulation of resistances for different failure modes is also conservative for cracked concrete.

Further research is also needed to examine the influence of modified edge distances $c_1$. For sure, the estimates for concrete edge failure will increase with larger edge distances $c_1$ (cf. equation (2)) just as the anchorage resistance will do with larger anchorage lengths, until the limiting steel failure is
reached. Nevertheless the question remains, if the experimental failure loads will rise as much as the calculated estimates will do.

6 Acknowledgement

The authors would like to thank Dr. Alawieh, Dr. Winkler and the whole team of KIBKON at Ruhr-Universität Bochum for their support in experimental testing.

References:


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8. EOTA: European Assessment Document EAD 330008-02-0601, Anchor Channels, 02/2016.
ANCHOR CHANNELS UNDER SHEAR LOAD ACTING IN THE LONGITUDINAL DIRECTION OF THE CHANNEL AXIS

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ABSTRACT

Anchor channels transfer tensile as well as shear forces acting perpendicular and in the longitudinal direction of the channel axis by means of mechanical interlock into the concrete. Furthermore, the load transfer of combined tension and shear loads is possible. A safe and efficient design of anchor channels requires detailed knowledge of the load-bearing behavior of the anchor channels depending on the load direction. Based on the PhD thesis of Kraus¹ covering the effect of tension loads and Potthoff² investigating the effect of shear forces acting perpendicular to the channel axis a design approach including combined tension loads was developed by addressing all actions to the anchors. Therefore, in the current standards differing approaches for the distribution of loads to the individual anchors of an anchor channel are given. A satisfactory approach considering the load distribution for anchor channels under shear load acting in the longitudinal direction of the channel axis is not yet developed.

This paper provides a new and satisfactory engineering model for the determination of the anchor forces and the load distribution of shear loads acting in the longitudinal direction of the channel axis based on numerical and experimental studies. This method consists of the application of a simplified load distribution model. The determination of the load distribution to the individual anchors of the channel is based on tuning factors representing the influence of e.g. parameters like anchor spacing, bolt position or edge distance. These factors are determined for the anchors under consideration and are multiplied with the acting load. This approach allows the determination of shear and tension loads acting on an anchor so that a verification of the interacting loads can be performed. Further information is given in the PhD thesis of Schmidt³.

1 Introduction

For anchor channels subjected to shear loads acting in the longitudinal direction of the channel axis no efficient design rules are given up to now. The approach given in Ref.⁴ to determine the load distribution to the individual anchors of the channel is shown in Figure 1. In this approach the external shear load is distributed to the “decisive” anchors, all remaining anchors are unloaded. This is maybe a conservative procedure and does not describe the real load distribution.
Examples of the numerical and experimental investigations which are described in Ref 3 are explained below. Furthermore, an engineering model for determining the anchor loads is outlined shortly.

2 Numerical investigations

2.1 General
In Ref.3 a comprehensive parametric study was carried out by means of finite element calculations. In addition to the profile size the geometric parameters like number of the anchors (n), anchor spacing (s), load position (b_p) or edge distance (c_l) were investigated.

It has been demonstrated, that due to a shear load in direction of the channel axis expected shear loads in the anchors as well as non-negligible tension loads were generated. Consequently a force model of an anchor channel section with one anchor can be derived, Figure 2.

The numerical results are evaluated by a stiffness criterion that describes the situation immediately after the occurrence of a defined first damage of the concrete (“damage level”). The results show the anchor loads at this damage level.

2.2 Effect of number of anchors
How the number of anchors (s) effects the load distribution of an anchor channel under shear load in the direction of the channel axis (V_x) is investigated on channels with two, three, four and five anchors. For a better comparison the anchor spacing is in all cases s = 200 mm. Furthermore, the load is positioned over the very first anchor (Figure 3).
The shear component of the first anchor is higher than the individual component of the remaining anchors. With increasing number of anchors the individual component of the remaining anchors is more uniform. This results from the higher stiffness of the channel system. Considering the tension component, due to the load position the second anchor is most stressed.

2.3 Effect of anchor spacing

In this part the effect of different anchor spacing to the load distribution is investigated. Anchor channels with anchor spacing \( s = 100 \text{ mm} \), \( s = 200 \text{ mm} \) and \( s = 300 \text{ mm} \) will be considered. The load is positioned over the first anchor \( (b_p = 0 \text{ mm}) \). The load component of the anchors as a consequence of the shear load in direction of channel axis is shown in Figure 4.
2.4 Effect of load position
The effect of the load position is exemplary shown for anchor channels with three anchors without influence of the concrete edge. The anchor spacing is \( s = 200 \text{ mm} \), the load is shifted from the front to the back anchor successively. According to Figure 5 a major influence of the load position is detectable. The shear load of the anchor directly in front of the load is higher than the shear load of the remaining anchors. Furthermore, independent of the load position the shear load of the last anchor is the lowest. Considering the tension loads, the anchor behind the load position is most stressed.

![Figure 5: Load components of an anchor channel with three anchors and different load positions](image)

2.5 Effect of edge distance
The effect of different edge distances to the load distribution on anchor channels with two anchors is exemplary shown in Figure 6.
The load distribution depends on the edge distances up to \( c_1 = 150 \text{ mm} \). For anchor channels with edge distances of more than 150 mm a constant load distribution can be identified. This transition area can be exemplified by the different failure modes. Edge distances below \( c_1 = 100 \text{ mm} \) provoke concrete edge failure from the second anchor. For greater edge distances than \( c_1 = 150 \text{ mm} \) result in concrete break out failure from the second anchor. In between mixed failure modes occur.

### 3 Engineering Model

#### 3.1 General

Based on the PhD thesis of Kraus\(^1\) covering the effect of tension loads and Potthoff\(^2\) investigating the effect of shear forces acting perpendicular to the channel axis a design approach was developed by addressing all actions to the anchors, Figure 7.

For a load distribution approach for shear loads in the direction of the channel axis the anchor loads has to be determined accordingly. A sketch is shown in Figure 8.
In the following sections the determination of the shear as well as the tension loads of the anchors will be explained.

### 3.2 Distribution of shear components

Out of the numerical results, the following essential conclusions for the distribution of the shear component can be made:

- The shear component of the last anchor is minimum (min)
- The shear component of the anchor under or in front of the acting load is maximum (max)

The determination of the load distribution to the single anchors of the channel is based on tuning factors representing the influence of e.g. parameters like anchor spacing, bolt position or edge distance. These factors are determined for the anchors under consideration and are multiplied with the acting load. The application of the tuning factors is shown in Figure 10.
The tuning factors are determined as follows:

\[ \psi_{av} = \frac{1}{n} \]  
\[ \psi_{max} = 1,15 \]  
\[ \psi_{\min} = 0,85 - 0,03 \cdot b_p \sum(s) \]  
\[ \psi_{c1} = \frac{1}{1,38 - 0,003 \cdot c_1} \leq 1,0 \]  
\[ \delta = \frac{\psi_{av}(3 - \psi_{max} - \psi_{\min} - \psi_{c1})}{n - 1} \]  

with

- \( n \) number of anchors [-]
- \( b_p \) load position [mm]
- \( s \) anchor distance [mm]
- \( c_1 \) edge distance [mm]

3.3 Distribution of tension components

As mentioned earlier, due to the acting shear load tension loads in the anchors are generated. The engineering model is based on the assumption that the total tension loads of all anchors are less than 40% of the acting shear load \( V_x^3 \). 25% of the acting shear load \( V_x \) will be distributed equally on all anchors (\( \psi_{z,av} \)). The remaining 15% of \( V_x \) are distributed to the most loaded anchors (\( \psi_{z,max} \)). The average and maximum tension loads are exemplarily shown in Figure 11.

![Figure 11: Average and maximum tension load distribution – examples](image)

The application of the tuning factors is shown in Figure 11.
The tuning factors can be determined as follows:

(6) \[
\psi_{z,av} = \frac{0.25}{n}
\]

(7) \[
\psi_{z,max} = 0.075
\]

(8) \[
\psi_{z,c1} = 1 - 0.98^c_1
\]

(9) \[
\delta_z = \frac{\psi_{z,av} + \psi_{z,max}A_1 - (\psi_{z,av} + \psi_{z,max}A_1) \cdot \psi_{z,c1}}{n - 1}
\]

with

- \( n \) Number of anchors [-]
- \( c_1 \) Edge distance [mm]

4 Conclusion

Based on numerical and experimental investigations considering anchor channels under a shear load in the direction of the channel axis an engineering model was developed\(^3\). The described approach allows the determination of shear and tension loads acting on an anchor of an anchor channel under shear load in the direction of the channel axis. Furthermore, a verification of the interacting tension and shear loads perpendicular to the channel axis and in its direction can be performed.

References:


EXPERIMENTAL STUDY OF FASTENINGS FOR CURTAIN WALL APPLICATIONS – COMPARISON BETWEEN HEADED ANCHORS, WELDED EMBEDS, BONDED ANCHORS AND ANCHOR CHANNELS

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2Hilti Corporation, Schaan, Liechtenstein
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ABSTRACT

A typical application in which fastening systems are installed close to edges of a concrete member is fastening of curtain wall facades. Due to the small edge distance the steel capacity of the fastening system cannot fully be utilized and the controlling failure mode is breakout of the concrete edge. According to current design standards (DIN EN 1992-4 1 and AC232 2) the calculated concrete breakout capacity of anchor channels yields to a lower failure capacity compared to headed anchors. Therefore, often the use of an anchor channel is not possible where headed anchors fulfil the requirements in design. An experimental test program was carried out to evaluate the difference in concrete breakout capacity between headed anchors and anchor channels installed parallel to the edge and loaded towards the free edge. For comparison, welded embeds (headed anchors welded to a steel plate embedded flush to the concrete surface) and bonded anchors were also tested. In all tests the same geometrical parameters (edge distance and anchor spacing of the fastening system) were chosen. The tests were performed in concrete of same mix composition, age and concrete strength. This allows a direct comparison of the test results. Based on the results of the experimental investigations the different design models are evaluated.

1 Introduction

According to current design standards (DIN EN 1992-4 1 and AC232 2) the calculated concrete breakout capacity of anchor channels yields to a lower resistance compared to headed anchors for same edge distance and member thickness. Main reasons for the reduced capacity of anchor channels are listed in the following:

- In design, for anchor channels an uneven load distribution to the anchors is assumed whereas for headed anchors the shear load is evenly distributed to the anchors.
- For anchor channels the critical spacing, critical member thickness and critical corner distance to ensure full breakout body are assumed to be much larger compared to headed
anchors. Whereas for headed anchors \( s_{cr,v} = 3c_1 \), \( h_{cr,v} = 1.5c_1 \) and \( c_{cr,v} = 1.5c_1 \), for anchor channels \( s_{cr,v} = 4c_1 + 2b_{ch} \), \( h_{cr,v} = 2c_1 + 2h_{ch} \) and \( c_{cr,v} = 2c_1 + b_{ch} \) need to be applied in design.

- The basic equation to calculate the concrete edge breakout resistance of one anchor is different for anchor channels and headed anchors.

To evaluate the difference in concrete edge breakout capacity between anchor channels and headed anchors, experimental investigations for a typical fastening application (\( n = 3 \) anchors, \( c = 100 \) mm, \( s = 150 \) mm) were performed. The tests are described in detail in Section 2. For the tested parameters, according to DIN EN 1992-4\(^1\), the fastening with anchor channels lead to 30% higher utilization in design compared to the fastening with headed anchors for the same acting shear load, whereas according to CEN/TS 1992-4\(^3\) headed anchors and anchor channels lead to comparable utilization in design. It is noted that a direct comparison of the calculated resistances is not possible since for headed anchors the resistance of the entire group is calculated whereas for anchor channels the most unfavourable anchor is verified. More details can be found in a special paper explaining the differences in DIN EN 1992-4\(^1\) between anchor channels and headed anchors (Grosser, Basche\(^4\)).

2 Experimental Investigation

A total of 20 tests performed at the Faculty of Civil Engineering in Rijeka, Croatia, are reported in this paper. Four types of fastening systems were tested in 300 mm thick concrete slabs. In addition, anchor channels were tested as well in 200 mm thick concrete slabs. Four tests per concrete slab and type of fastening system were carried out. The main goal of the research was to investigate the difference in concrete shear breakout capacity for the different types of fasteners arranged close to the edge of a concrete member. To make the results comparable, all tests were performed in concrete slabs of same mix composition, age and concrete strength. In addition, the same geometrical parameters such as edge distance and anchor spacing were used.

2.1 Fastening systems

The experimental program included testing of four types of fastening systems: headed anchors (Figure 1a), welded embeds (Figure 1b), anchor channels (Figure 1c) and bonded anchors (Figure 1d). Each fastener group consisted of 3 anchors. All anchors in test series CS1 and CS2 had a diameter of 16 mm with a steel grade 8.8. Detailed specifications for tested anchor channels HAC-50 together with channel bolts HBC-C M16 (CS3 and CS4) can be found in the Hilti Anchor Channel HAC Technical Guide\(^5\). Bonded anchors (CS5) were cut to 160 mm length from 16 mm diameter threaded meter rods (steel grade 8.8) and installed in holes of hardened concrete using HY200 adhesive.
2.2 Concrete Slabs

Five concrete slabs were casted, four of them with dimensions $1600 \times 1600 \times 300$ mm and one with dimensions $1600 \times 1600 \times 200$ mm. Normal weight low strength concrete (strength class C20/25) was used for all concrete specimens. The concrete was produced according to the requirements of HRN EN 2016-1:2006 and HRN 1128:2007. Concrete mixture was kept constant for all concrete slabs. Concrete consistency was determined as S2 and maximum aggregate size of 16 mm was used. All tested slabs were reinforced with wire-mesh Q 131 embedded near the top and bottom of concrete slabs for handling purposes (see Figure 2). Due to the wire position (approximately 200 mm away from the edges of the slab), the reinforcement did not affect the outcome of the test results. All slabs were cast horizontally and compacted using a vibrator. The concrete specimens were stored in the laboratory at the Faculty of Civil Engineering, Rijeka, and wrapped with plastic sheets according to standard HRN EN 12390-2:2009. After 28 days the plastic sheets were removed and concrete slabs were stored at ambient temperature until the day of testing.

Three cubes with a side length of 150 mm were cast for each concrete slab. Concrete compressive strength was determined in accordance with the European standards HRN EN 12504-1:2009 and HRN EN 12390-3:2009 at the Faculty of Civil Engineering, Rijeka. The concrete cubes were cured in a water tank for 28 days in accordance with HRN EN 12390-2:2009 and afterwards cured in air until testing. The measured compressive strength at the time of testing scattered between 31.22 MPa and
44.78 MPa (average 38.33 MPa). However, to more accurately represent the concrete strength of the tested specimens, three concrete cylinders with diameter of 100 mm and height of 100 mm were cored from each concrete slab after completion of the tests. The compressive strength measured on concrete cores is converted into the cube strength using Eq. 2.1e of ETAG 001, Annex A\(^9\). The average concrete compressive strength based on the results of 15 tested cores is 33.61 MPa (results ranged from 31.28 MPa to 36.97 MPa).

### 2.3 Installation of fastening systems

The installation of the fastening systems was done in accordance with the requirements defined in the installation instructions. The first 3 types of fasteners (headed anchors, welded embeds and anchor channels) were cast into concrete, while bonded anchors were post-installed in the hardened concrete slab. All cast-in systems were installed under laboratory conditions surface-flush into the formwork. In case of post-installed anchors, the holes were drilled using a rotary hammer drill HILTI TE 4-A22 with drill bit of 18 mm diameter. After drilling, the holes were cleaned following the installation instructions using a steel wire brush and compressed air. The adhesive HIT-HY 200-A was used to fill the holes and to fix the threaded rods by pushing them into the adhesive. Prior to testing, curing time of approximately 24 hours was kept.

Each fastening system consisted of three anchors spaced in a distance of 150 mm. In all tests, anchors were arranged parallel to the edge of the concrete slabs and subjected to shear loading acting perpendicular towards the edge. All anchors were installed with an edge distance of 100 mm. Headed anchors, welded embeds and bonded anchors were installed with an embedment depth of 102 mm, anchor channels with an embedment depth of 106 mm. Detailed installation parameters of each fastening system are summarized in Table 1. Figure 3 illustrates an example of a concrete specimen.

<table>
<thead>
<tr>
<th>Concrete slab ID</th>
<th>CS1</th>
<th>CS2</th>
<th>CS3</th>
<th>CS4</th>
<th>CS5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastener group</td>
<td>headed anchors</td>
<td>welded embeds</td>
<td>anchor channels</td>
<td>anchor channels</td>
<td>bonded anchors</td>
</tr>
<tr>
<td>Slab thickness, (h) (mm)</td>
<td>300</td>
<td>300</td>
<td>300</td>
<td>200</td>
<td>300</td>
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<tr>
<td>Edge distance, (c) (mm)</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>Anchor spacing, (s) (mm)</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>Embedment depth (mm)</td>
<td>102</td>
<td>102</td>
<td>106</td>
<td>106</td>
<td>102</td>
</tr>
<tr>
<td>Diameter of anchors (mm)</td>
<td>16</td>
<td>16</td>
<td>9</td>
<td>9</td>
<td>16</td>
</tr>
<tr>
<td>Height of channel profile or welded steel plate (mm)</td>
<td>-</td>
<td>30</td>
<td>31</td>
<td>31</td>
<td>-</td>
</tr>
<tr>
<td>Width of channel profile or welded steel plate (mm)</td>
<td>-</td>
<td>40</td>
<td>41.9</td>
<td>41.9</td>
<td>-</td>
</tr>
<tr>
<td>Net edge distance (mm)</td>
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<td>80</td>
<td>79</td>
<td>79</td>
<td>-</td>
</tr>
<tr>
<td>Overlap of channel profile or welded steel plate (mm)</td>
<td>-</td>
<td>25</td>
<td>25</td>
<td>25</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 3: Concrete slab CS3 – Tests with anchor channels: top and side view (all dimensions in mm)

2.4 Test Setup and Procedure

The tests were performed in the laboratory of the Faculty of Civil Engineering in Rijeka on a strong floor equipped with ZWICK ROELL hydraulic actuator, type LH 0250-100. To ensure a fix position of the concrete slab during testing, a special steel frame was designed and manufactured. Figure 4 shows the strong floor and position of steel frame and concrete slab. The steel frame was placed in front of the hydraulic actuator and mounted to the strong floor with steel rods (M20, 8.8 CS). Horizontal support on the front side is shown in Figure 4a. The concrete slabs were placed on top of the steel frame and mounted as shown in Figure 4b to avoid lift up during testing.

(a) (b)

Figure 4: Strong floor and steel frame for testing fastening systems in shear (a) detail of fixing steel frame to strong floor and (b) detail of fixing concrete slab to steel frame

The test setup for shear tests is shown in more detail in Figure 5. For all tests, spacing of support blocks was sufficient to ensure no influence on concrete breakout cone. Figure 6 and 7 show details of the loading plate, actuator and connection between fastening system and loading plate. The loading plate was connected to the servo hydraulic actuator with a threaded rod (M20, 8.8 CS). Depending on the tested fastening system, the loading plate was connected directly to the anchors or
channel bolts (hole diameter in plate 18 mm) or connected with M20 adapter bolts in case of welded embeds. A torque moment of 60 Nm was applied. A 2 mm thick ptfe sheet was placed between concrete and loading plate to reduce friction.

![Figure 5: Test setup](image)

The displacement was measured using displacement transducers type LD 320-50 OMEGA. Two LVDTs were glued to the top of the concrete surface behind the outermost anchors. The location of the LVDTs can be seen in Figure 7. The LVDTs measured the horizontal displacement in the direction of loading. The anchor displacement is determined by averaging two LVDT measurements. Anchor shear load was measured by a load cell, type BPS-TL0250.10.00 (max load 250 kN) (see Figure 5).

![Figure 6: Schematic sketch of loading plate (all units in mm)](image)
The tests were performed at room temperature. The age of concrete at time of testing was approximately 2 months. Four replicates were performed per test series. The load was applied by controlling the hydraulic actuator displacement at a constant displacement rate of 0.02 mm/s such that the peak load was reached in approximately 3 to 5 minutes. Load and displacement measurements were recorded with sampling rate of 100 Hz and collected via data acquisition system (National Instruments).

3 Experimental Results

The test results (measured ultimate load and corresponding displacement at ultimate load) for each test series are summarized in Table 2.

Table 2: Summary of shear test results

<table>
<thead>
<tr>
<th>Concrete slab ID</th>
<th>CS1 $\text{(h=300 mm)}$</th>
<th>CS2 $\text{(h=300 mm)}$</th>
<th>CS3 $\text{(h=300 mm)}$</th>
<th>CS4 $\text{(h=200 mm)}$</th>
<th>CS5 $\text{(h=300 mm)}$</th>
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</thead>
<tbody>
<tr>
<td>Fastening system</td>
<td>Headed anchors</td>
<td>Welded embeds</td>
<td>Anchor channels</td>
<td>Anchor channels</td>
<td>Bonded anchors</td>
</tr>
<tr>
<td>Test series</td>
<td>Shear load (kN)</td>
<td>Shear load (kN)</td>
<td>Shear load (kN)</td>
<td>Shear load (kN)</td>
<td>Shear load (kN)</td>
</tr>
<tr>
<td>1</td>
<td>45.82</td>
<td>49.54</td>
<td>44.58</td>
<td>37.21</td>
<td>57.95</td>
</tr>
<tr>
<td>2</td>
<td>48.15</td>
<td>53.77</td>
<td>41.83</td>
<td>38.13</td>
<td>51.33</td>
</tr>
<tr>
<td>3</td>
<td>46.45</td>
<td>54.48</td>
<td>45.62</td>
<td>37.28</td>
<td>50.24</td>
</tr>
<tr>
<td>4</td>
<td>46.97</td>
<td>45.44</td>
<td>44.72</td>
<td>37.18</td>
<td>48.50</td>
</tr>
<tr>
<td>Mean value$^1$</td>
<td>46.85</td>
<td>50.81</td>
<td>44.19</td>
<td>37.45</td>
<td>52.00</td>
</tr>
<tr>
<td>$f_c$ [MPa]</td>
<td>26.52</td>
<td>28.62</td>
<td>26.00</td>
<td>26.02</td>
<td>27.28</td>
</tr>
</tbody>
</table>

Table 2: Summary of shear test results

<table>
<thead>
<tr>
<th>Concrete slab ID</th>
<th>CS1 $\text{(h=300 mm)}$</th>
<th>CS2 $\text{(h=300 mm)}$</th>
<th>CS3 $\text{(h=300 mm)}$</th>
<th>CS4 $\text{(h=200 mm)}$</th>
<th>CS5 $\text{(h=300 mm)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fastening system</td>
<td>Headed anchors</td>
<td>Welded embeds</td>
<td>Anchor channels</td>
<td>Anchor channels</td>
<td>Bonded anchors</td>
</tr>
<tr>
<td>Test series</td>
<td>Displacement at ultimate shear load (mm)</td>
<td>Displacement at ultimate shear load (mm)</td>
<td>Displacement at ultimate shear load (mm)</td>
<td>Displacement at ultimate shear load (mm)</td>
<td>Displacement at ultimate shear load (mm)</td>
</tr>
<tr>
<td>1</td>
<td>0.81</td>
<td>3.16</td>
<td>1.70</td>
<td>0.71</td>
<td>0.84</td>
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<tr>
<td>2</td>
<td>0.99</td>
<td>2.49</td>
<td>1.31</td>
<td>0.74</td>
<td>0.88</td>
</tr>
<tr>
<td>3</td>
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<td>2.10</td>
<td>1.17</td>
<td>0.82</td>
<td>1.33</td>
</tr>
<tr>
<td>4</td>
<td>0.81</td>
<td>2.14</td>
<td>1.19</td>
<td>0.60</td>
<td>0.76</td>
</tr>
<tr>
<td>Mean value$^1$</td>
<td>0.94</td>
<td>2.47</td>
<td>1.34</td>
<td>0.72</td>
<td>0.95</td>
</tr>
</tbody>
</table>

$^1$Mean value is the average of four tests in a series.
Figure 8: Load-displacement curves for tested fastening systems
Figure 8 and Table 2 show that the ultimate strength of headed anchors and anchor channels is approximately the same in 300 mm thick concrete slabs. Compared to headed anchors and anchor channels, the tests with welded embeds show ~10% higher ultimate strength. Interestingly, it was observed that with bonded anchors the measured ultimate strength is also about 10% higher compared to headed anchors. The measured displacement at ultimate strength of anchor channels, and especially that of welded embeds, is larger than that of headed anchors and bonded anchors. In terms of ductility, it is observed that headed anchors and bonded anchors exhibit relatively brittle post-peak response whereas the post-peak response of anchor channels and welded embeds is more ductile.

Typical breakout pattern for each type of fastening system are shown in Figure 9. As expected, the breakout pattern is similar in all tests. However, it is observed that the crack starts from the steel plate or channel profile in case of welded embeds (CS2) and anchor channels (CS3 and CS4) which leads to a smaller breakout body due to the smaller net edge distance ($c' = c - b_{ch}/2$).

![Typical breakout pattern for each type of fastening system](image-url)
Comparison with Current Standards

To allow a direct comparison between the tested fastening systems, all test results were normalized to a concrete compressive strength $f_c = 25$ MPa (Figure 10a). The related failure loads (measured ultimate strength to mean ultimate strength calculated from test series CS1) are shown in Figure 10b. Anchor channels tested in a 300 mm thick concrete slab show a slightly lower ultimate strength compared to headed anchors. This can be explained by the smaller anchor diameter for anchor channels. M16 anchors are used for headed anchors and welded embeds, whereas the anchor diameter of the anchor channels is only 9 mm. According to DIN EN 1992-4\textsuperscript{1} for an edge distance of 100 mm, the concrete breakout strength of a 9 mm anchor is $\sim10\%$ lower compared to the concrete breakout strength calculated for an anchor with a diameter of 16 mm. The tests with welded embeds show $\sim5\%$ higher ultimate strength compared to headed anchors. The steel plate of the welded embeds has the same dimensions (width, height and length) as the channel profile of the anchor channel. Therefore, the welded embed can be considered as an anchor channel with infinite rigidity.

![Figure 10: (a) Failure loads normalized to $f_c = 25$ MPa and (b) ratio between failure loads and mean failure load of test series CS1](image)

Current design standards (DIN EN 1992-4\textsuperscript{1} and AC232\textsuperscript{2}) take into account the influence of the member thickness in case of anchor channels by the reduction factor $\psi_{ch,h,V}$. In case of the tested anchor channels in a 200 mm thick concrete slab with 100 mm edge distance, Equation 1 leads to a reduction of the calculated concrete breakout strength of $\sim15\%$. This represents well the measured reduction observed in tests.

$$\psi_{ch,h,V} = \left(\frac{h}{h_{cr,V}}\right)^{0.5} \leq 1.0 \quad \text{with} \quad h_{cr,V} = 2c_1 + 2h_{ch} \quad (1)$$

According to DIN EN 1992-4\textsuperscript{1} there should be no difference between headed anchors and bonded anchors. However, tests with bonded anchors show a 10\% higher ultimate strength in shear compared to headed anchors.
5 Conclusions

Main purpose of the research described in this paper was to investigate the difference in concrete breakout strength of anchor channels and headed anchors. According to DIN EN 1992-4\textsuperscript{1} for the investigated parameters, the design of anchor channels for the concrete edge failure mode leads to 30\% higher utilization compared to the design of headed anchors whereas according to CEN/TS 1992-4\textsuperscript{3} headed anchors and anchor channels lead to comparable utilization in design. The research described in this paper shows that the breakout strength in shear of anchor channels is comparable to headed anchors for the same parameters. Planning engineers now face the challenge how a cost-efficient solution for anchor channels can be realized with design according to DIN EN 1992-4\textsuperscript{1} and AC232\textsuperscript{2}. The discrepancy between headed anchors and anchor channels needs to be eliminated in design.

6 Acknowledgement

The authors are grateful to the construction company GP Krk d.d., Tehnoplast Profili d.o.o. and Laboratory at Faculty of Civil Engineering in Rijeka for their support. Financial and technical support was provided by Hilti AG. The authors would also like to thank AS Schoeler + Bolte GmbH for providing the headed anchors and welded embeds specially designed for this research project.

References:


DETERMINATION OF CONCRETE BREAKOUT OF CONCRETE BREAKOUT STRENGTH FOR CAST-IN CHANNELS WITH MULTIPLE ANCHORS EXPOSED TO SHEAR LOADS

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ABSTRACT

Serrated anchor channels provide reliable, durable and adjustable connections to concrete. In common applications, anchor channels are located close to the edge and can be loaded in tension and in shear in longitudinal direction of the channel axis or perpendicular to the channel axis, which is of special interest in this study. According to recent design guides¹,² the concrete breakout strength is influenced by the edge distance, compressional concrete strength, neighboring anchors, member thickness, corner effects, and shear forces acting on the channel. Applying several meters length of anchor channel equipped with multiple anchors, one realizes that in a conservative case where the load transmitting bolts are installed above each anchor, the influence of neighboring anchors decreases the resistance of the anchor currently under consideration significantly. This assumption is on the conservative. However, this does not reflect the physical response of long embedded anchor channels realistically. To encourage a discussion in this field and to demonstrate the need for further research, numerical simulations with finite element method³ were scheduled. Previous physical experiments, carried out at the Structural Testing Laboratory at the Ruhr-University Bochum, were used to calibrate the numerical model. The numerical simulations were performed on long anchor channels with a varying number of anchors, exposed to shear loading above each anchor towards the edge. The influence of the concrete breakout strength was evaluated numerically and compared with the analytical assumptions¹.

1 Introduction

Anchor channels are an ideal solution for adjustable and easy-to-install connections. C-shaped steel channel profiles with serrated lips are merged with round anchors riveted to the channel back. The anchor channels are cast in concrete. After the concrete has hardened arbitrary constructions can be attached with channel bolts, and adjusted (a posteriori). No drilling or welding is necessary for the attachment; this avoids damage to the concrete and the reinforcement. Anchor channels have to pass a complex certification process and meet required testing criteria to be granted an approval. The outstanding approved product is an optimization in respect to static and economic demands. In the past,
approval tests were carried out on anchor channels with two anchors. This procedure is
experimentally appropriate and at the same time cost-effective. In real applications anchor channels
can range up to a length of several meters implying an increasing number of anchors. In terms of
measurement instrumentation, it is very complicated to setup an experiment for anchor channels with
multiple anchors and to apply shear loads over a multiple of channel bolts simultaneously. The
concrete resistance by means of concrete breakout failure for such long anchor channels was
typically determined according to analytical guidelines\textsuperscript{1,2}. Above each anchor one serrated
hammerhead channel bolt (T-bolt) was installed. The decisive anchor was verified. Although strong
dehformations occur in the channel profile no load transmission is assumed over the channel profile
into the concrete; the shear load is fully distributed over the anchors into the concrete. The analytical
formulas in current guidelines\textsuperscript{1,2} consider that anchors within a base length influence each other
resulting in a factor to reduce the basic characteristic resistance of the concrete breakout strength. On
the action side a linear load distribution caused by each installed bolt is assumed. When bolts are
installed within the influence lengths of neighboring bolts the load triangles superimpose and
increase the load acting on the anchor. As a consequence, the anchor resistance of the decisive
anchor is reduced due to the influence of neighboring anchors and exposed to an enlarged action.
This conservatism emphasizes the product safety and reliability but is disadvantageous to the
manufacturer. In the following numerical study we want to illustrate the need for further research on
anchor channels of meters in length and encourage discussion on the current design guides.

2 Problem statement

According to FprEN 1992-4\textsuperscript{1} the characteristic resistance of the concrete breakout failure of one
anchor loaded perpendicular to the edge is defined as

\begin{equation}
V_{Rk,c} = V_{Rk,c}^0 \cdot \psi_{ch,s,V} \cdot \psi_{ch,c,V} \cdot \psi_{ch,r,V} \cdot \psi_{ch,90°,V},
\end{equation}

where $V_{Rk,c}^0$ is the basic characteristic resistance and $\psi_{ch,s,V} = f(s_i; s_{cr,V}; V_i/V_0)$, $\psi_{ch,c,V} =
\psi(c_2; c_{cr,V})$ and $\psi_{ch,r,V} = f(h; h_{cr,V})$ are reduction factors to account for the influence of
neighboring anchors, a corner and the concrete member thickness on the concrete edge resistance,
respectively. Since the presented study discusses the influence of the edge distance $c_1$ ($c_2 \to \infty$) and
anchor channels exposed to shear loading perpendicular to the channel axis $V_{Ed} = V_{Ed,y}$, the
reduction factor

\begin{equation}
\psi_{ch,s,V} = \frac{1}{1 + \sum_{i=1}^{n_{ch,V}} \left(1 - \frac{s_i}{s_{cr,V}}\right)^{1.5} \cdot \frac{V_i}{V_0}}
\end{equation}

is of special interest. $V_i$ and $V_0$ are the shear forces acting on the influencing anchors and the anchor
under consideration, respectively (see Figure 1). The critical anchor spacing is defined as is of
$s_{cr,V} = 4 \cdot c_1 + 2 \cdot b_{ch} \cdot n_{ch,V}$ indicates the number of anchors within the distance $s_{cr,V}$ to be
considered as influencing.
In general for anchor channels with multiple anchors, the shear resistance $V_{Rd}$ is reduced by $\psi_{ch,s,V}$ due to the influence of neighboring anchors. The shear action in each anchor $V_{Ed,i}^{a} (= V_i)$ caused by a shear load acting on the channel, $V_{Ed}$, can be calculated according to

$$V_{Ed,i}^{a} = k \cdot A_i' \cdot V_{Ed}.$$  \hspace{1cm} (3)$$

The Eq.(3) assumes a linear load distribution over the influence length, $l_i$, and takes into account the condition of equilibrium. The ordinate $k = 1/\sum A_i'$ with $A_i' = (l_i - s)/l_i$ is located at position of the anchor $i$ of a triangle with the unit height at position of the load $V_{Ed}$ and the base length $2 \cdot l_i$. The influence length $l_i = 13 \cdot I_y^{-0.05} \cdot s^{0.5}$ can be calculated with the moment of inertia of the channel relative to y-axis, $I_y$, and anchor spacing $s$. To illustrate the conservatism mentioned before the reduction factors $\psi_{ch,s,V,i}$, which affect the concrete breakout strength and the amplification $k \cdot A_i'$ of the acting shear load, are illustrated in Table 1 for anchor channels with two (2A) through to eight anchors (8A). Calculating the reduction factors and the amplification for the anchor under consideration analytically (for input details see Section°3.1), it can be seen in Table 1 that $\psi_{ch,s,V}$ is converging towards the value of 0.399 and the maximum of $\sum k \cdot A_i'$ towards the value of 1.046 for anchor channels with more than three anchors.

<table>
<thead>
<tr>
<th>Number of anchors</th>
<th>2A</th>
<th>3A</th>
<th>4A</th>
<th>5A</th>
<th>6A</th>
<th>7A</th>
<th>8A</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\psi_{ch,s,V}$ (resistance)</td>
<td>0.619</td>
<td>0.482</td>
<td>0.399</td>
<td>0.399</td>
<td>0.399</td>
<td>0.399</td>
<td>0.399</td>
</tr>
<tr>
<td>max $[\sum k \cdot A_i']$ (action)</td>
<td>1.000</td>
<td>1.092</td>
<td>1.046</td>
<td>1.046</td>
<td>1.046</td>
<td>1.046</td>
<td>1.046</td>
</tr>
</tbody>
</table>

Table 1: Evaluation of $\psi_{ch,s,V}$ and $k \cdot A_i'$ according to FprEN 1992-4 for the most unfavorable anchor under consideration for anchor channels with two anchors (2A) through to eight anchors (8A)
This approach is on the conservative, but it does not necessarily reflect the physical behavior of the cast-in anchor channels. For anchor channels with multiple anchors the resistance may be underestimated with the result that the capacity of anchor channels by means of the concrete breakout strength is not fully utilized.

3 Experiments

3.1 Physical experiments

In four physical experiments one type of serrated channel profiles with riveted round anchors was used. The profile’s clearance was $b_{ch} \times h_{ch} = 52.5 \times 34$ mm. The anchor channels were cast in a plain concrete slab of dimensions $2000 \times 2000 \times 250$ mm. The anchor channels with a maximum anchor spacing $s_{\text{max}} = 250$ mm were arranged parallel to the edge at a distance $c_1 = 200$ mm without any corner influence. The load was transmitted with tension plates over T-bolts and anchor channel into the concrete (see Figure 2, left). A layer of Polytetrafluoroethylene was placed between the tension plate and channel surface to minimize friction. The slab was supported on a stiff horizontal and vertical construction located on the strong floor of the Structural Testing Laboratory at the Ruhr-University Bochum (KIBKON). As suggested by AC232 the support reaction was located at a distance $\geq 2.5 \cdot c_1$ to the anchors to ensure undisturbed crack formation. To record the post-peak behavior the specimen was loaded until local collapse of concrete. Diagonal cracks formed a breakout body that was pushed by the strongly deformed channel lips. The enlarged area of failure can be seen in Figure 2 (right).

![Figure 2: Experimental setup for shear testing of the anchor channel 53/34 (left) and failed specimen after concrete breakout (right)](image-url)
3.2 Numerical simulations

The global model was composed of one concrete and several steel parts. The concrete material was modeled using the Concrete Damage Plasticity (CDP) model implemented in ABAQUS. The implementation details of the model were adapted from Lubliner et al. The concrete specimen was meshed with 10-node quadratic tetrahedron elements (see Figure 3, left). The steel anchor channel, bolts and washers were meshed with 8-node linear brick elements with reduced integration and hourglass control (see Figure 3, left and right). In order to provoke concrete failure the steel parts were modeled being elastic. Tangential and normal contact was assumed in the interface between the concrete and steel parts. The supports and load application were simplified by rigid bodies to save computational memory. The numerical simulation was performed displacement-controlled. The material properties are listed in Table 2. The parallel numerical simulations were computed on the Euler High-Performance Cluster of ETH Zurich using the Newton-Raphson-Solver of ABAQUS/Standard.

Table 2: Material parameters for FEM simulations with ABAQUS

<table>
<thead>
<tr>
<th>Concrete</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{\text{cm,cube}}$</td>
<td>$f_{\text{cm}}$</td>
</tr>
<tr>
<td>[N/mm$^2$]</td>
<td>[N/mm$^2$]</td>
</tr>
<tr>
<td>29.4</td>
<td>1.87</td>
</tr>
</tbody>
</table>

4 Results

4.1 Model calibration

The composite model exhibited different material properties. In addition, all parts with individual contact laws were in contact governed by constrains in material and surface roughness. In order to approximate the physical behavior of the composite structure properly by means of a finite element approach, each material model needed to be validated with physical experiments. Calibration experiments were carried out on all parts. For details on the calibration of the anchor channel failure see Kocur et al. The physical experiments previously mentioned were utilized to calibrate the CDP model. As can be observed in Figure 4 (left) the computed load-displacement curve closely matches
the experimental curves with respect to the failure load. The close similarity is further supported by
the simulated post-peak cracking patterns that appear similar to the experimental patterns (see
Figure 4, right). The calibration setup of the anchor channel with two anchors was adapted as a
reference for further simulations with multiple anchors.

Figure 4: Comparison of experimental and numerical failure loads (left) and post-peak cracking
patterns from experiment with finite element simulation (right)

4.2 Influence of neighboring anchors

Additional numerical simulations of anchor channels of the type 53/34 with multiple anchors were
carried out. The anchor channels were embedded in a concrete model of height $h$ and edge distance
$c_1$. The setup was chosen identical to the reference calculation to ensure comparability and to carve
out the influence of the reduction factor $\psi_{ch,s,V}$ and the amplification $k \cdot A_i'$ solely. The number of
anchors simulated ranged from three through to eight anchors riveted at the distance $s_{\text{max}}$ used
before.

Figure 5: Load-displacement curves computed with FEM for anchor channels with two through to
eight anchors (left). Influence of the number of anchors on the normalized reduction factors $\psi_{ch,s,V}$
and amplification $F_{u,y}/F_{u,y}^{2A}$ (right)
Only the number of anchors was varied; the material parameters and boundary conditions were kept unchanged. Figure 5 (left) illustrates the load-displacement curves and failure loads computed with ABAQUS. The verification for the concrete breakout failure was performed with respect to the most unfavorable anchor. On the conservative, the total shear load acting on the anchor channel is transmitted through the anchors into the surrounding concrete. The computed failure loads were distributed equally over all anchors referred to as \( F_{\text{u},y} \). On the action side, the neighboring anchors take over the fraction \( \Sigma k \cdot A_i' \) of the shear load \( V_{Ed} \). The ratio \( F_{u,y} / F_{u,y}^{2A} \) needs to be divided by \( \max[\Sigma k \cdot A_i'] \) to obtain equally transmitted shear loads in the anchors \( V_{Ed}^a \). The numerically determined ratio between the ultimate load \( F_{u,y} \) and the reference load \( F_{u,y}^{2A} \) was compared with the ratio of the reduction factors \( \psi_{ch,s,v} / \psi_{ch,s,v}^{2A} \) on the resistance side. Note that this assumption is a simplification. At that time no reliable measurement of effective anchor forces in anchor channels can be found in literature.

5 Discussion and Conclusions

The close similarity of load-displacement curves displayed in Figure 5 (left) shows that the CDP model implemented in ABAQUS is well suited to simulate composite structures such as performed in the current study. However, due to lack of experimental data, physical experiments on anchor channels with more than three anchors should be done in future studies.

In this particular case (\( s_{\max} = 250 \text{ mm} \)) five anchors in total (\( i = 0 \cdots 4 \)) influence each other (\( s_{ct,v} = 905 \text{ mm} \)). Analytically, the additional anchors (> 4A) will have a minor effect on the concrete breakout strength, respectively on the reduction factors. This behavior can be observed in Table 1 from 5A on. Since the case with two anchors serves as the reference the influence of loading and resistance is assumed as being 1.0 and their difference is set to zero. For three through eight anchors the difference ranges from 7.1% (3A) to 24.3% (4A); this is equivalent to anchor loads ranging from 3.0 kN to 10.5 kN, respectively. For five through eight anchors the difference seems to vanish. In Figure 5 (right), it is interesting to observe that the difference jumps for four anchors. From an economic point of view this case is the most disadvantageous. Further simulations accompanying by physical experiments with varying \( s_{\max} \) and \( c_1 \) should by carried out and analyzed with respect to the reduction factor \( \psi_{ch,s,v} \). At the same time the effective loads in the anchors computed should be compared to the linear load distribution approach over the influence length.

It can be clearly observed that the ratio of failure loads marked with dots in Figure 5 (right) is underestimated by the analytical reduction factors marked with diamonds. There is potential for optimization on the action side as well as on the resistance side if the reduction factor and the load distribution taken into account are not too conservative.

6 Acknowledgements

Eleni Chatzi of the Institute of Structural Engineering, ETH Zurich is gratefully acknowledged for supporting the project with expertise on numerical modeling and simulations with ABAQUS.
References:


4. Kocur, G. K., Chatzi, E. and Häusler, F. The structural behavior of serrated cast-in anchor channels – a numerical study on the longitudinal loading close to the edge In Proceedings of the 9th International Conference on Fracture Mechanics of Concrete and Concrete Structures (FraMCoS-9), Berkeley, CA, USA, May 29 – June 1, 2016.
ENHANCEMENT OF THE CALCULATION METHOD FOR ANCHOR CHANNELS TO GROUPS OF ANCHOR CHANNELS

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ABSTRACT

Fastenings with anchor channels are a current method of anchorages in concrete. The length of the channel and the number of anchors are not limited, only the spacing of the anchors has to comply with the specified minimum spacing. With respect to further anchor channels in the concrete member there is the restriction that the spacing between two anchors of neighboring anchor channels shall not be less than the characteristic spacing $s_{cr,N}$. This spacing ensures the characteristic resistance of one anchor in case of concrete cone failure under tension load. So anchorages with anchor channels are limited to single-row fastenings.

Headed fasteners or post-installed fasteners with a common fixture may be arranged in groups with up to three rows with three anchors each. There is no restriction regarding the distance between the rows of anchors within the group. So these fasteners allow for multi-row anchorages. A requirement comparable with anchor channels is that the spacing between the outer anchor of adjoining groups or a single anchor shall be greater than the characteristic spacing $s_{cr,N}$, with $s_{cr,N}=3\cdot h_{ef}$.

If there is the necessity to connect one fixture to two anchor channels with parallel alignment overly large attachments may be the consequence. But also for anchor channels arranged in a row to enable continuous fastening along the concrete member the restriction of the spacing between anchors of different channels may cause problems. Furthermore the required spacing makes the arrangement of two anchor channels at a corner of a concrete member difficult.

In order to overcome this restriction in use and enable the use of pairs or groups of anchor channels with a spacing between two anchors of different channels smaller than the characteristic value $s_{cr,N}$ a proposal for the enhancement of the existing equations to calculate the capacity for concrete cone breakout under tension load is presented.

1 Introduction

The scope of FprEN 1992-4¹ and EOTA TR 047² or ICC-ES AC232³ which cover the design of anchor channels do not address the length of the channels, the number of anchors or the required spacing between two anchors of different anchor channels in the case that more than one is placed in the concrete member. According to the European assessment document EAD⁴ the number of anchors on an anchor channel is not limited, only the spacing between the anchors has to be constant and
shall not fall below the specified minimum spacing $s_{\text{min}}$. The length of an anchor channel is limited by production only. The requirement regarding the distance between anchors of different channels is stated in the section “Intended use”: “The distance between two or more anchor channels is in such a way that no spacing between two anchors of neighboring anchor channels is less than $s_{cr,N}$.” ICC-ES AC232 which is also an assessment document does not address the distance between adjoining anchor channels.

The characteristic spacing is defined as $s_{cr,N}=2-(2.8-1.3 \cdot h_{ef}/180) \cdot h_{ef}$, where $h_{ef}$ is the effective embedment depth. For current anchor channels the characteristic spacing is between $5 \cdot h_{ef}$ and $3 \cdot h_{ef}$. This leads characteristic spacings $s_{cr,N}$ between $225 \text{mm}$ for $h_{ef}=45 \text{mm}$ and $540 \text{mm}$ for $h_{ef}=180 \text{mm}$.

A group of headed fasteners or post-installed fasteners with a common fixture may comprise up to three rows of anchors with a maximum of three anchors in each row. The characteristic spacing for this fasteners is in all cases $s_{cr,N}=3 \cdot h_{ef}$, and thus usually smaller than for anchor channels. Additionally the distance between two rows of anchors may be less than $s_{cr,N}$. A fixture with four or six fasteners could be considerably smaller than a fixture to a pair of anchor channels with four or six channel bolts.

For several anchor channels arranged in a row the required spacing $s_{cr,N}$ between the last anchor of one channel and first anchor of the following channel causes a large gap where no fastening is possible. Fastenings with two anchor channels across a corner of the concrete member are affected in the same extend. The mentioned situations with multiple anchor channels are shown in Figure 1.

\begin{figure}[h]
\centering
\includegraphics[width=0.5\textwidth]{figure1.png}
\caption{Arrangement of multiple anchor channels}
\end{figure}
2 Design Approach

The characteristic resistance of a single anchor of an anchor channel in case of concrete cone failure under tension load is calculated according to Equation (1)\textsuperscript{1,2}:

\[ N_{Rk,c} = N_{Rk,c}^0 \cdot \psi_{ch,s,N} \cdot \psi_{ch,e,N} \cdot \psi_{ch,c,N} \cdot \psi_{re,N} \] (1)

There is no verification for a group of anchors but for each single anchor. The factor \( \psi_{ch,s,N} \) as defined in Equation (2) considers the influence of neighboring anchors. The other factors in Equation (1) are not relevant at this examination.

\[ \psi_{ch,s,N} = \frac{1}{1 + \sum_{i=1}^{n_{ch,N}} \left( \frac{s_i}{s_{cr,N}} \right)^{1.5} \cdot \frac{N_i}{N_0}} \] (2)

For several anchor channels arranged in a row there is no reason to require a spacing \( s_{cr,N} \) between last and first anchor of adjoining channels. Self-evident the forces in the anchors due to tension loads acting on the channels have to be calculated separately for each channel. The second channel is basically an extension of the first channel and the influence of anchors of the second channel may be considered directly with Equation (2). The only requirement is that the distance between the anchors complies with the minimum spacing between two anchors according to the technical specification.

A possible approach to respect the influence of anchors of an adjacent anchor channels on an anchor of the channel under consideration could be the assumption of a virtual edge between the profiles. The influence of the anchors of the neighboring anchor channel would be considered by the factor \( \psi_{ch,e,N} \) according Equation (3).

\[ \psi_{ch,e,N} = \left( \frac{c_1}{c_{cr,N}} \right)^{0.5} \leq 1.0 \] (3)

This may work with anchor channels with parallel alignment because here position and orientation of the virtual edge are clearly defined. With perpendicular arrangement of the channels maybe different virtual edges have to be assumed depending on which anchor is under consideration. Another shortcoming of this approach is the non-consideration of different loads on the anchors.

Here it is proposed instead to use generally the factor \( \psi_{s,N} \) to account for the influence of adjoining anchor channels or other fastenings. This factor accounts for the distance of the influencing anchors to the anchor under consideration and the tension loads acting on the anchor.

For parallel aligned identical anchor channels with a spacing \( s_{ch} \) Equation (2) has to be amended to respect the influence of the anchors of the second channel on the anchor under consideration of the first channel. The formulation becomes more clearly arranged if Equation (2) is rewritten in that way as shown in Equation (4). The index “\( i \)” starts here at the value 0, and the anchor under consideration is regarded as an influencing anchor with the spacing \( s_0=0 \) mm.
To be used for pairs of identical anchor channels with the distance \(s_{ch}\) between the axis of the channels the amendment of Equation (4) results in Equation (5) which determines the influence of neighboring anchors of both anchor channels on the anchor 0 of anchor channel 1.

\[
\psi_{ch,s,N,i} = \frac{1}{\sum_{i=0}^{n_{ch,N}} \left(1 - \frac{s_i}{s_{cr,N}}\right)^{1.5} \cdot \frac{N_i}{N_0}}
\]

To demonstrate that this approach is usable the results of calculations with the amended factor are compared with the figures which are obtained from the calculations if the anchor channels are expected to be anchor groups.

The characteristic resistance of a group of fasteners under tension loading in case of concrete cone failure is determined according to Equation (6)\(^1\).

\[
N_{Rk,c} = N_{Rk,c}^0 \cdot \frac{A_{c,N}}{A_{c,N}^0} \cdot \psi_{s,N} \cdot \psi_{te,N} \cdot \psi_{ec,N} \cdot \psi_{M,N}
\]

For the comparison only the ratio \(A_{c,N}/A_{c,N}^0\) and the factor \(\psi_{ec,N}\) are of interest. The ratio of the actual projected area \(A_{c,N}\) and the reference projected area \(A_{c,N}^0\) accounts for the geometric effect of axial spacing of the anchors. The characteristic spacing \(s_{cr,N}\) of anchors and anchor channels with identical effective embedment depth \(h_{ef}\) may differ but are postulated to be identical here. The reference area is determined by Equation (7).

\[
A_{c,N}^0 = s_{cr,N}^2
\]

The actual projected area depends on the number of anchors \(n_a\) in a row or the number of anchors of the anchor channel respectively, and the number of rows of anchors or the number \(n_{ch}\) of parallel anchor channels respectively. The actual projected area may be obtained from Equation (8), with the spacing of the anchors \(s\) and the spacing of the anchor channels \(s_{ch}\).

\[
A_{c,N} = \left((n_a - 1) \cdot s + s_{cr,N}\right) \cdot \left((n_{ch} - 1) \cdot s_{ch} + s_{cr,N}\right)
\]

The factor \(\psi_{ec,N}\) takes into account the effect on the anchor group if different tension loads are acting on the single anchors and is determined according to Equation (9).

\[
\psi_{ec,N} = \frac{1}{1 + \frac{e_{N,x}}{s_{cr,N}}/s_{cr,N}} \cdot \frac{1}{1 + \frac{e_{N,y}}{s_{cr,N}}/s_{cr,N}}
\]

The eccentricity \(e_{N,x}\) respects different loads on the anchors of one channel, and \(e_{N,y}\) different loads on corresponding anchors of the different channels. Both factors in the product must not be taken greater than the value 1.
The maximum capacity of a group of fasteners may be expressed by Equation (10).

\[ N_E \leq N_{R,c}^0 \cdot \frac{A_{c,N}^0}{A_{c,N}^0} \cdot \psi_{ec,N} \]  

(10)

The capacity of each anchor of an anchor channel may be described by Equation (11)

\[ N_E^a \leq N_{R,c}^0 \cdot \psi_{ch,s,N} \]  

(11)

The load \( N_E^a \) on an anchor is only a portion of the total load \( N_E = \sum N_E^a \) acting on a single channel or a pair of channels. To make the resistances of groups of fasteners and anchor channels comparable Equation (11) is rewritten and refers to the total load as given in Equation (12). The capacity of the anchor channel is governed by the anchor with the minimum value \( \psi_{ch,s,N}/(N_E^a/N_E) \).

\[ N_E \leq N_{R,c}^0 \cdot \min \left( \frac{\psi_{ch,s,N}}{N_E^a/N_E} \right) \]  

(12)

The calculations are performed for anchor channels with two and three anchors. The spacing of the anchors is \( s=250 \text{ mm} \), the effective embedment depth \( h_{ef}=155 \text{ mm} \). The distance between the anchor channels in case of pairs is taken as \( s_{ch}=250 \text{ mm} \). The input values as loading and eccentricity and the results of the calculations are shown in Tables 1 to 7.

### Table 1: 2 anchors without load eccentricity

<table>
<thead>
<tr>
<th>Anchor</th>
<th>( x_a )</th>
<th>( N^a/N )</th>
<th>( \sum(1-s/s_{ch,N})^{1.5}N/N_0 )</th>
<th>( \psi_{ec,N} )</th>
<th>( \psi_{s,N}/(N_E^a/N_E) )</th>
<th>( N_E^a/N_{R,c}^0 )</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0.500</td>
<td>1.38</td>
<td>-</td>
<td>0.73</td>
<td>1.45</td>
<td>0.98</td>
</tr>
<tr>
<td>2</td>
<td>250</td>
<td>0.500</td>
<td>1.38</td>
<td>-</td>
<td>0.73</td>
<td>1.45</td>
<td></td>
</tr>
</tbody>
</table>

### Anchor group

<table>
<thead>
<tr>
<th>( A_{c,N}^0 )</th>
<th>( A_{c,N} )</th>
<th>( c_{ec,N,x} )</th>
<th>( \psi_{ec,N,x} )</th>
<th>( c_{ec,N,y} )</th>
<th>( \psi_{ec,N,y} )</th>
<th>( \psi_{ec,N} )</th>
<th>( A_{c,N}/A_{c,N}^0 \cdot \psi_{ec,N} )</th>
<th>( N_E^a/N_{R,c}^0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>271412</td>
<td>401655</td>
<td>0.00</td>
<td>1.00</td>
<td>-</td>
<td>-</td>
<td>1.00</td>
<td>1.48</td>
<td>1.48</td>
</tr>
</tbody>
</table>

### Table 1: 2 anchors with load eccentricity

<table>
<thead>
<tr>
<th>Anchor</th>
<th>( x_a )</th>
<th>( N^a/N )</th>
<th>( \sum(1-s/s_{ch,N})^{1.5}N/N_0 )</th>
<th>( N_E^a/N_{R,c}^0 )</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0.750</td>
<td>1.13</td>
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<td>250</td>
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<td>2.13</td>
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<td>0.47</td>
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### Anchor group

<table>
<thead>
<tr>
<th>( A_{c,N}^0 )</th>
<th>( A_{c,N} )</th>
<th>( c_{ec,N,x} )</th>
<th>( \psi_{ec,N,x} )</th>
<th>( c_{ec,N,y} )</th>
<th>( \psi_{ec,N,y} )</th>
<th>( \psi_{ec,N} )</th>
<th>( A_{c,N}/A_{c,N}^0 \cdot \psi_{ec,N} )</th>
<th>( N_E^a/N_{R,c}^0 )</th>
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</thead>
<tbody>
<tr>
<td>271412</td>
<td>401655</td>
<td>62.50</td>
<td>0.81</td>
<td>-</td>
<td>0.81</td>
<td>1.19</td>
<td>1.19</td>
<td>1.19</td>
</tr>
</tbody>
</table>
### Table 3: 2x2 anchors with load eccentricity in two directions

<table>
<thead>
<tr>
<th>Anchor channel</th>
<th>Ratio</th>
</tr>
</thead>
</table>
| Anchor | $x_a$ | $N^a_{1/N}$ | $\sum(1-s/s_{cr})^{1.5}N/N_0$ | $N^a_{2/N}$ | $\sum(1-s/s_{cr})^{1.5}N/N_0$ | $\psi_{s,N}$ | $\psi_{s,N}/(N^a_{E}/N^a_{E})$ | $N_0/E_{Re}$ |$
1$ | $0$ | 0.563 | 1.13 | 0.188 | 0.15 | 0.79 | 1.40 | 1.40 |
| 2 | 250 | 0.188 | 2.13 | 0.063 | 0.31 | 0.41 | 2.19 | 0.98 |

| Anchor group | $A_{c,N}$ | $A_{c,N}$ | $\psi_{ec,N,x}$ | $\psi_{ec,N,y}$ | $\psi_{ec,N}$ | $A_{c,N}/A^0_{c,N}/\psi_{ec,N}$ | $N_0/E_{Re}$ |$
271412$ | 594398 | 62.5 | 0.81 | 62.5 | 0.81 | 0.65 | 1.42 | 1.42 |

### Table 4: 3x1 anchors without load eccentricity

<table>
<thead>
<tr>
<th>Anchor channel</th>
<th>Ratio</th>
</tr>
</thead>
</table>
| Anchor | $x_a$ | $N^a_E$ | $\sum(1-s/s_{cr})^{1.5}N/N_0$ | $N^a_{2/N}$ | $\sum(1-s/s_{cr})^{1.5}N/N_0$ | $\psi_{s,N}$ | $\psi_{s,N}/N^a_{E}$ | $N_0/E_{Re}$ |$
1$ | $0$ | 0.333 | 1.38 | - | - | 0.72 | 2.17 | 1.71 |
| 2 | 250 | 0.333 | 1.75 | - | - | 0.57 | 1.71 | 0.87 |
| 3 | 500 | 0.333 | 1.38 | - | - | 0.72 | 2.17 | 0.87 |

| Anchor group | $A_{c,N}$ | $A_{c,N}$ | $\psi_{ec,N,x}$ | $\psi_{ec,N,y}$ | $\psi_{ec,N}$ | $A_{c,N}/A^0_{c,N}/\psi_{ec,N}$ | $N_0/E_{Re}$ |$
271412$ | 531898 | 0.0 | 1.00 | - | - | 1.00 | 1.96 | 1.96 |

### Table 5: 3x2 anchors without load eccentricity

<table>
<thead>
<tr>
<th>Anchor channel</th>
<th>Ratio</th>
</tr>
</thead>
</table>
| Anchor | $x_a$ | $N^a_{1/N}$ | $\sum(1-s/s_{cr})^{1.5}N/N_0$ | $N^a_{2/N}$ | $\sum(1-s/s_{cr})^{1.5}N/N_0$ | $\psi_{s,N}$ | $\psi_{s,N}/(N^a_{E}/N^a_{E})$ | $N_0/E_{Re}$ |$
1$ | $0$ | 0.167 | 1.38 | 0.167 | 0.56 | 0.52 | 3.09 | 2.41 |
| 2 | 250 | 0.167 | 1.75 | 0.167 | 0.74 | 0.40 | 2.41 | 0.83 |
| 3 | 500 | 0.167 | 1.38 | 0.167 | 0.56 | 0.52 | 3.09 | 0.83 |

| Anchor group | $A^0_{c,N}$ | $A_{c,N}$ | $\psi_{ec,N,x}$ | $\psi_{ec,N,y}$ | $\psi_{ec,N}$ | $A_{c,N}/A^0_{c,N}/\psi_{ec,N}$ | $N_0/E_{Re}$ |$
271412$ | 787141 | 0.0 | 1.00 | 0.00 | 1.00 | 1.00 | 2.90 | 2.90 |

### Table 6: 3x2 anchors with load eccentricity in one direction

<table>
<thead>
<tr>
<th>Anchor channel</th>
<th>Ratio</th>
</tr>
</thead>
</table>
| Anchor | $x_a$ | $N^a_{1/N}$ | $\sum(1-s/s_{cr})^{1.5}N/N_0$ | $N^a_{2/N}$ | $\sum(1-s/s_{cr})^{1.5}N/N_0$ | $\psi_{s,N}$ | $\psi_{s,N}/(N^a_{E}/N^a_{E})$ | $N_0/E_{Re}$ |$
1$ | $0$ | 0.250 | 1.38 | 0.083 | 0.19 | 0.64 | 2.55 | 2.00 |
| 2 | 250 | 0.250 | 1.75 | 0.083 | 0.25 | 0.50 | 2.00 | 0.86 |
| 3 | 500 | 0.250 | 1.38 | 0.083 | 0.19 | 0.64 | 2.55 | 0.86 |

| Anchor group | $A^0_{c,N}$ | $A_{c,N}$ | $\psi_{ec,N,x}$ | $\psi_{ec,N,y}$ | $\psi_{ec,N}$ | $A_{c,N}/A^0_{c,N}/\psi_{ec,N}$ | $N_0/E_{Re}$ |$
271412$ | 787141 | 0.0 | 1.00 | 62.5 | 0.81 | 0.81 | 2.34 | 2.34 |
Table 7: 3x2 anchors with load eccentricity in two directions

<table>
<thead>
<tr>
<th>Anchor channel</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Anchor</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>x_1</td>
<td>0.375</td>
</tr>
<tr>
<td>N_{a_1}/N</td>
<td>1.25</td>
</tr>
<tr>
<td>\sum(1-s/s_{cr,N})/N_{a_1}</td>
<td>0.125</td>
</tr>
<tr>
<td>N_{a_1}</td>
<td>0.17</td>
</tr>
<tr>
<td>\psi_{a_1,N}</td>
<td>0.71</td>
</tr>
<tr>
<td>\psi_{a_1,N}/(N_{E}/N_{E})</td>
<td>1.88</td>
</tr>
<tr>
<td>N_{E}/N_{E}Re</td>
<td>1.06</td>
</tr>
<tr>
<td>x_2</td>
<td>0.250</td>
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<tr>
<td>N_{a_2}/N</td>
<td>1.75</td>
</tr>
<tr>
<td>\sum(1-s/s_{cr,N})/N_{a_2}</td>
<td>0.083</td>
</tr>
<tr>
<td>N_{a_2}</td>
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</tr>
<tr>
<td>\psi_{a_2,N}</td>
<td>0.50</td>
</tr>
<tr>
<td>\psi_{a_2,N}/(N_{E}/N_{E})</td>
<td>2.00</td>
</tr>
<tr>
<td>N_{E}/N_{E}Re</td>
<td></td>
</tr>
<tr>
<td>x_3</td>
<td>0.125</td>
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<tr>
<td>N_{a_3}/N</td>
<td>1.77</td>
</tr>
<tr>
<td>\sum(1-s/s_{cr,N})/N_{a_3}</td>
<td>0.042</td>
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<tr>
<td>N_{a_3}</td>
<td>0.25</td>
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<tr>
<td>\psi_{a_3,N}</td>
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<tr>
<td>\psi_{a_3,N}/(N_{E}/N_{E})</td>
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<tr>
<td>N_{E}/N_{E}Re</td>
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</tr>
</tbody>
</table>

Anchor group

<table>
<thead>
<tr>
<th>A_{c,N}</th>
<th>A_{c,N}</th>
<th>e_{c,N,x}</th>
<th>\psi_{c,N,x}</th>
<th>e_{c,N,y}</th>
<th>\psi_{c,N,y}</th>
<th>\psi_{c,N}</th>
<th>A_{c,N}/A_{c,N}^0</th>
<th>\psi_{c,N}</th>
<th>N_{E}/N_{E}Re</th>
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<tbody>
<tr>
<td>271412</td>
<td>787141</td>
<td>83.3</td>
<td>0.76</td>
<td>62.5</td>
<td>0.81</td>
<td>0.61</td>
<td>1.77</td>
<td>1.77</td>
<td>1.77</td>
</tr>
</tbody>
</table>

3 Conclusion

The resistances of single anchor channels with two anchor and pairs of anchor channels with two and three anchors in case of concrete cone failure under tension load is calculated with the models for anchor channels and groups of fasteners. The comparison of the two methods to calculate the resistance of pairs of anchor channels shows a well to satisfying match of the results. For channels with two anchors or pairs of channels with two anchors the determined resistances are practically identical.

The largest deviations occur at anchor channels with three anchors where the anchors of one channel are loaded equally. Here the capacity of the anchor channel is up to 17% smaller than that of a group of fasteners with the same anchor layout. This results from the verification of the single anchor at anchor channels in contrast to the verification of the entire group of fasteners.

The approach to consider the influence of additional anchor channels on the resistance of an anchor in case of concrete cone failure by means of the factor \psi_{ch,s,N} seems to be feasible. There is no need to keep the requirement that the spacing between two anchors of adjoining channels shall not be less than s_{cr,N}. Further considerations are necessary if the anchor channels are loaded in shear. It is imaginable to apply the procedure as for fasteners. In case of pairs of anchor channels both channels may be assumed to resist concrete pryout-failure, and the channel closest to the edge only to resist concrete edge failure. The arrangement of two anchor channels across the corner of the concrete member may need further considerations to determine the resistance.

References:

ANCHOR CHANNELS WITH SHORT EMBEDMENT DEPTH USED IN COMPOSITE SLAB CONSTRUCTION

Philipp Grosser 1*, Teodora Dimitrova 1, Bernhard Winkler 1

1 Hilti Corporation, Schaan, Liechtenstein
*Corresponding Author Email: philipp.grosser@hilti.com

ABSTRACT

The use of anchor channels in concrete construction is well accepted in practice due to their high load-bearing capacity and flexibility in adjustment after installation. Transfer of external tension loads back into the concrete member is achieved by mechanical interlock between the headed anchor connected to the channel profile and the concrete. Standard anchor channels covered by European and US approvals have a ratio $h_{ch}/h_{ef} \leq 0.4$ and $b_{ch}/h_{ef} \leq 0.7$ ($h_{ch}$ = height of the channel profile, $b_{ch}$ = width of the channel profile and $h_{ef}$ = embedment depth). Anchor channels with $h_{ch}/h_{ef} > 0.4$ and/or $b_{ch}/h_{ef} > 0.7$ are only covered in current design standards (e.g. DIN EN 1992-41 and AC2322) with the option to reduce the embedment depth to the length of the anchor underneath the channel profile since the influence of the channel profile on the concrete breakout capacity is not sufficiently investigated. However, the use of anchor channels with short embedment depth is increasing in composite slab construction. Composite slabs comprise reinforced concrete cast on top of profiled steel decking. The thickness of the concrete over the trapezoidal deck profile is often smaller than the nominal anchor length of standard anchor channels. The research described in the paper provides information about the concrete breakout capacity of anchor channels with reduced embedment depth. Based on the results of the investigations a design recommendation is given to calculate the concrete breakout resistance more accurately.

1 Introduction

In case of headed studs, the concrete cone breakout capacity of a centrically loaded group in unreinforced concrete without influence of concrete edges ($N_{u,c}$ according to Eq. (1)) is calculated by multiplying the basic equation of an uninfluenced single anchor $N_{u,c}^0$ (Eq. (2)) with the ratio of the projected area.

$$N_{u,c} = N_{u,c}^0 \cdot \frac{A_{c,N}}{A_{c,N}^0} \quad [N]$$  \hspace{1cm} (1)

$$N_{u,c}^0 = 15.5 \cdot \sqrt{f_{cc,200}} \cdot h_{ef}^{1.5} \quad [N]$$  \hspace{1cm} (2)
with:

- $h_{ef}$: effective embedment depth
- $f_{cc,200}$: concrete cube compressive strength [N/mm²] at the time of testing (cube with side length of 200 mm)
- $s$: center-to-center spacing of headed anchors

$$\frac{A_{c,N}^0}{A_{c,N}} = 1 + \frac{s}{s_{cr,N}} \quad \text{(in case of n = 2 anchors without influence of a concrete edge (s ≤ s_{cr,N}))}$$

$$s_{cr,N} = 3 \cdot h_{ef} \quad \text{(characteristic anchor spacing to ensure the failure load of an individual anchor in case of concrete cone failure)}$$

In case of anchor channels, in design, the verification is based on the most unfavorable individual anchor and not, as in case of headed studs, on the verification of the group. However, for an anchor channel with two anchors centrally loaded in unreinforced concrete without influence of concrete edges, the breakout capacity can be displayed in the same format provided in Eq. (1) for headed anchors. For anchor channels where $h_{ch}/h_{ef} ≤ 0.4$ and $b_{ch}/h_{ef} ≤ 0.7$, the breakout capacity $N_{u,c}$ can be calculated according to Eq. (3). Compared to headed anchors, the basic equation of an uninfluenced single anchor $N_{u,c}^0$ (Eq. (4)) is reduced by the influence of the channel profile on the concrete cone breakout capacity (Eq. (5)). The influence of neighboring anchors on the concrete cone breakout capacity is taken into account by the factor $\psi_{ch,s,N}$, which is calculated according to Equation (6). Also, the characteristic spacing is assumed to be larger compared to headed anchors (Eq. (7)).

$$N_{u,c} = N_{u,c}^0 \cdot \psi_{ch,s,N} \quad [N]$$  

$$N_{u,c}^0 = 15.5 \cdot \sqrt{f_{cc,200} \cdot h_{ef}^{1.5} \cdot \psi_{ch,N}} \quad [N]$$  

with:

- $h_{ef}$: effective embedment depth according to Figure 1a)
- $s, f_{cc,200}$: according to Eq. (1)

$$\psi_{ch,N} = \left( \frac{h_{ef}}{180} \right)^{0.15} \leq 1.0$$  

516
\[
\psi_{ch,s,N} = \frac{2}{1 + \left(1 - \frac{s}{s_{cr,N}}\right)^{1.5}}
\]

(in case of \(n = 2\) anchors without influence of a concrete edge (\(s \leq s_{cr,N}\))

\[
s_{cr,N} = \left(5.6 - 2.6 \cdot \frac{h_{ef}}{180}\right) \cdot h_{ef} \geq 3 \cdot h_{ef}
\]

(characteristic anchor spacing to ensure the failure load of an individual anchor in case of concrete cone failure)

\begin{align*}
\text{a) } & h_{ef} \text{ for } h_{ch}/h_{ef} \leq 0.4 \text{ and } b_{ch}/h_{ef} \leq 0.7 \\
\text{b) } & h_{ef}^* \text{ for } h_{ch}/h_{ef} > 0.4 \text{ and/or } b_{ch}/h_{ef} > 0.7
\end{align*}

Figure 1: Current definition of embedment depth for anchor channels installed in concrete

In case of \(h_{ch}/h_{ef} > 0.4\) and/or \(b_{ch}/h_{ef} > 0.7\), the concrete cone breakout capacity may be calculated using one of the following options:

- the effective embedment depth is determined according to Figure 1b), \(h_{ef} = h_{ef}^*\), or
- the effective embedment depth \(h_{ef}\) is determined according to Figure 1a) but the value for \(s_{cr,N}\) shall be taken from the relevant Technical Product Specification (e.g. ETA or ICC-ESR). The value for \(s_{cr,N}\) used in design shall not be smaller than that for anchor channels where \(h_{ch}/h_{ef} \leq 0.4\) and \(b_{ch}/h_{ef} \leq 0.7\) is fulfilled (see Equation (7)).

2 Research significance

Table 1 shows the dimensions \((h_{ef}, h_{ch}, b_{ch})\) and the geometrical ratios \(h_{ch}/h_{ef}\) and \(b_{ch}/h_{ef}\) for the Hilti anchor channels (HAC) covered in the current ETA and ICC-ESR. All channels fulfill the requirement \(h_{ch}/h_{ef} \leq 0.4\) and \(b_{ch}/h_{ef} \leq 0.7\). This also applies to anchor channels from other companies available on the market. For simplification, such anchor channels are named “standard anchor channels” in the following.
Table 1: Anchor channels (HAC) currently available on the market and minimum possible embedment depth fulfilling $h_{ch}/h_{ef} \leq 0.4$

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Units</th>
<th>Anchor channel sizes</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>HAC-30</td>
</tr>
<tr>
<td>Embedment depth</td>
<td>$h_{ef}$</td>
<td>(mm)</td>
<td>68</td>
</tr>
<tr>
<td>Channel height</td>
<td>$h_{ch}$</td>
<td></td>
<td>25.6</td>
</tr>
<tr>
<td>Channel width</td>
<td>$b_{ch}$</td>
<td></td>
<td>41.3</td>
</tr>
<tr>
<td>Ratio channel height to embedment depth</td>
<td>$h_{ch}/h_{ef}$</td>
<td>-</td>
<td>0.38</td>
</tr>
<tr>
<td>Ratio channel width to embedment depth</td>
<td>$b_{ch}/h_{ef}$</td>
<td></td>
<td>0.61</td>
</tr>
<tr>
<td>Min. possible $h_{ef}$</td>
<td>$h_{ef,min}$</td>
<td>(mm)</td>
<td>64</td>
</tr>
</tbody>
</table>

The design provisions for the concrete cone failure mode are based on limited research performed by Kraus (2003). Even though the test results have not shown a negative influence of the channel profile on the concrete cone breakout capacity (see Figure 2), the reduction factor $\psi_{ch,N}$ according to Eq. (5) was still introduced in existing design standards.

Figure 2: Failure load $N_{u,25}$ of headed anchors and anchor channels (Kraus (2003))

When designing standard anchor channels, typically steel failure of the channel is the decisive failure mode under tension loading. Since concrete failure does not govern the design, there is no need to improve the provisions for concrete cone failure. However, with growing demands for lightweight and slimmer concrete construction, anchor channels are needed in thin concrete members. Often, the member thickness of the concrete structure is $h = 100$ mm. In composite slab construction, the remaining thickness of the concrete over the metal deck is even less ($h \sim 75$ mm). In this case, due to the reduced embedment depth needed, concrete cone breakout often controls design, especially for...
bigger profile sizes. The purpose of the research presented in this paper is to provide information about the following aspects:

- The limitations $h_{ch}/h_{ef} \leq 0.4$ and $b_{ch}/h_{ef} \leq 0.7$ are set arbitrarily based on limited research. The limiting value for $h_{ch}/h_{ef}$ is further studied. The limiting value $b_{ch}/h_{ef}$ is not further investigated since a factor of 0.7 is not exceeded for common anchor channels on the market.

- The reduction factor $\psi_{ch,N}$ taking into account the influence of the channel profile on the concrete cone breakout capacity according to Eq. (5) only depends on the embedment depth. The reduction of the concrete area due to different values $h_{ch}$ is not considered. This aspect is further discussed in this paper.

- Since the factor $\psi_{ch,N}$ already takes into account the influence of the channel profile on the concrete cone breakout capacity, it is not understandable why a larger critical spacing $s_{cr,N}$ is needed compared to headed studs. The larger influence factor is explained by Kraus (2003) with the disturbance of the stress distribution due to the channel profile. The correlation between $\psi_{ch,N}$ and $\psi_{ch,s,N}$ is discussed in this paper.

3 Experimental investigations

A test program was carried out with single headed anchors with 4 different embedment depths (60 mm, 80 mm, 100 mm and 120 mm). In order to guarantee concrete cone failure before failing the steel in tension, a steel grade 12.9 was used. The headed anchors were made of M16 threaded rods machined down to 13 mm core diameter to realistically represent typical anchor diameters. For each embedment depth 6 tests were performed, 3 in plain concrete and 3 with polystyrene foam around the upper part on the anchor ($H = h_{ch} = 40$ mm, $B = b_{ch} = 45.4$ mm) (see Figure 3). The polystyrene foam should simulate the influence of the anchor channel profile on the concrete cone breakout capacity. Testing with anchor channels currently available on the market would not result for all embedment depth in concrete cone failure due to lower steel resistance of the channel. The distance between the headed anchors was taken in all cases larger than $4h_{ef}$ to avoid an influence between adjacent anchors. The polystyrene foam was casted along the entire length of the concrete specimen which considers an anchor channel with infinite values for overlap $x$ and anchor spacing $s$. As an example, the test specimen for $h_{ef} = 80$ mm is shown in Figure 4.

![Figure 3: Headed anchor and installation of polystyrene foam in formwork](image3)
More details to the tests can be found in the test report\textsuperscript{4}. The measured failure loads are shown in Figure 5 for both the tested single headed anchors and the simulated anchor channels. For comparison, the failure loads calculated according to Equation (2) and Equation (4) are shown.

![Figure 6: Ratio between failure loads measured in tests and calculated failure loads](image)

In Figure 6 the ratio between failure loads measured in tests to calculated failure loads according to Equation (2) is plotted as a function of the ratio $\frac{h_{ef}}{h_{ef}}$. For comparison, the reduction factor $\psi_{ch,N}$ according to Equation (5) is shown.
Up to a ratio $h_{ch}/h_{ef} = 0.5$ the influence of the channel profile on the concrete cone breakout capacity is $\leq 10\%$. In tests with $h_{ef} = 60\text{ mm}$ ($h_{ch}/h_{ef} = 0.67$) a reduction of about $55\%$ was observed. This can be explained by the change of the failure mode (see Figure 7). For $h_{ch}/h_{ef} \leq 0.5$ concrete cone failure with cracking propagating up to the concrete surface was observed, whereas for $h_{ch}/h_{ef} = 0.67$ only a concrete breakout cone underneath the simulated channel profile was observed.

![Figure 6: $\psi_{ch,N}$ plotted as a function of the ratio $h_{ch}/h_{ef}$](image)

4 Numerical investigations

During the last 20 years, Hilti Corporation has developed a simulation tool based on the finite element method and concrete material models to understand the behavior of fastening systems in concrete. The numerical simulation includes the modeling of the fastening, the concrete specimen, as well as contact models between anchor and concrete. The simulation requires a realistic model of concrete including the cracking process after reaching the tensile strength of the material. The concrete material is modeled using a rotating crack model for the tensile zone and a plasticity model for the compressive regime, both of which are necessary in the regions of load transfer into the base.
material. The normal and tangential penetration is constrained by using penalties considering friction and bond, respectively. In the context of this investigation an elasto-plastic material model is used to describe the nonlinear material behavior of steel. The simulation tool based on these models has been proven very robust. The finite element simulations described in this paper comprise single headed anchors, anchor groups and anchor channels with one and two anchors (Figure 8). The investigated parameters are shown in Table 2. Both headed anchors and anchor channels were simulated with 8 different embedment depths in a range from 60 mm to 180 mm. The spacing of the anchors was varied from \( s = 0 \) (single anchor) up to \( s = 250 \) mm. Two different anchor channel profiles with rectangular shape were simulated. Type 1 equals HAC-50 and type 2 equals HAC-70 in terms of height and width of the channel profile.

Table 2: Finite element simulations (n = 120) - Investigated parameters

<table>
<thead>
<tr>
<th></th>
<th>( h_{ef} ) [mm]</th>
<th>( h_{ch} ) [mm]</th>
<th>( b_{ch} ) [mm]</th>
<th>( s ) [mm]</th>
<th>( h_{ch}/h_{ef} )</th>
<th>( b_{ch}/h_{ef} )</th>
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</thead>
<tbody>
<tr>
<td>Headed anchors</td>
<td>60, 70, 80, 90, 100, 120, 150, 180</td>
<td>-</td>
<td>-</td>
<td>0, 100, 150, 200, 250</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Anchor channels (type 1)</td>
<td>60</td>
<td>31</td>
<td>41.9</td>
<td>0, 100, 150, 200, 250</td>
<td>0.52</td>
<td>0.70</td>
</tr>
<tr>
<td></td>
<td>70</td>
<td>31</td>
<td>41.9</td>
<td>0, 100, 150, 200, 250</td>
<td>0.44</td>
<td>0.60</td>
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<td></td>
<td>80</td>
<td>31</td>
<td>41.9</td>
<td>0, 100, 150, 200, 250</td>
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<td>0.52</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>31</td>
<td>41.9</td>
<td>0, 100, 150, 200, 250</td>
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<td>0.47</td>
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<tr>
<td></td>
<td>100</td>
<td>31</td>
<td>41.9</td>
<td>0, 100, 150, 200, 250</td>
<td>0.31</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>120</td>
<td>31</td>
<td>41.9</td>
<td>0, 100, 150, 200, 250</td>
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<tr>
<td></td>
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<td>41.9</td>
<td>0, 100, 150, 200, 250</td>
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<tr>
<td></td>
<td>180</td>
<td>31</td>
<td>41.9</td>
<td>0, 100, 150, 200, 250</td>
<td>0.17</td>
<td>0.23</td>
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<tr>
<td>Anchor channels (type 2)</td>
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<td>40</td>
<td>45.4</td>
<td>0, 100, 150, 200, 250</td>
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<td>0.76</td>
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<tr>
<td></td>
<td>70</td>
<td>40</td>
<td>45.4</td>
<td>0, 100, 150, 200, 250</td>
<td>0.57</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>80</td>
<td>40</td>
<td>45.4</td>
<td>0, 100, 150, 200, 250</td>
<td>0.50</td>
<td>0.57</td>
</tr>
<tr>
<td></td>
<td>90</td>
<td>40</td>
<td>45.4</td>
<td>0, 100, 150, 200, 250</td>
<td>0.44</td>
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<td></td>
<td>100</td>
<td>40</td>
<td>45.4</td>
<td>0, 100, 150, 200, 250</td>
<td>0.40</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td>120</td>
<td>40</td>
<td>45.4</td>
<td>0, 100, 150, 200, 250</td>
<td>0.33</td>
<td>0.38</td>
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<tr>
<td></td>
<td>150</td>
<td>40</td>
<td>45.4</td>
<td>0, 100, 150, 200, 250</td>
<td>0.27</td>
<td>0.30</td>
</tr>
<tr>
<td></td>
<td>180</td>
<td>40</td>
<td>45.4</td>
<td>0, 100, 150, 200, 250</td>
<td>0.22</td>
<td>0.25</td>
</tr>
</tbody>
</table>

a) Simulations with one anchor: \( x = s_{cr,N}/2 \) acc. to Eq. (7)
b) Simulations with two anchors: $x = 25$ mm

Figure 8: Finite element model

In Figure 9 the ratio between failure loads of anchor channels to failure loads of headed anchors with one anchor is plotted as a function of the embedment depth and the ratio $h_{ch}/h_{ef}$. For comparison, the reduction factor $\psi_{ch,N}$ according to Equation (5) is shown. It can be seen in Figure 9a) that up to a ratio $h_{ch}/h_{ef} = 0.4$, the channel profile does not have a negative influence on the concrete cone breakout capacity. Between $0.4 < h_{ch}/h_{ef} \leq 0.5$ the disturbance of the strain and stress distribution, respectively, causes a reduction of the concrete cone breakout capacity in the order of 10% to 20%. In the numerical simulations with $h_{ch}/h_{ef} > 0.5$, in analogy to the findings in the experimental investigations described in Section 3, a change of the crack pattern is observed (see Figure 10). For $h_{ch}/h_{ef} = 0.5$ the concrete breakout cone is only developed underneath the channel profile.

The reduction factor $\psi_{ch,N}$ according to Eq. (5) only depends on the embedment depth. In Figure 9 it can be seen that the height of the channel profile $h_{ch}$ needs to be taken into account to realistically calculate the reduction of the concrete cone breakout capacity. The reduction observed in the numerical simulations is represented by the dashed line ($2-2.5h_{ch}/h_{ef}$).
Figure 10: Crack pattern observed in numerical simulations (left) and related capacity in an anchor group plotted as a function of the ratio between anchor spacing and embedment depth
a) Headed anchors b) Anchor channels (type 1) c) Anchor channels (type 2)
In Figure 10 the crack pattern of the simulations with headed anchors and both anchor channel types 1 and 2 are shown on the left-hand side. For the sake of transparency only embedment depth 60 mm, 100 mm and 150 mm are shown. It can be seen that the crack pattern, representing the diverse concrete cones, are influenced by the channel profile with reduced embedment depth. This effect is more pronounced with increasing height of the channel profile. On the right-hand side, the related capacity in an anchor group (ratio between simulation with two anchors and simulation with one anchor) is plotted as a function of the ratio between anchor spacing and embedment depth. For headed anchors, the increase of capacity in the group agrees well with the calculation approach according to Equation 1. The critical spacing is three to four times the embedment depth (Figure 10a).

According to Equation 7, the critical spacing for anchor channels depends on the embedment depth. For $h_{ef} = 180$ mm, the critical spacing $s_{cr,N}$ equals $3h_{ef}$, whereas for $h_{ef} = 60$ mm, the critical spacing $s_{cr,N}$ equals $4.7h_{ef}$. In Figure 10b) and c), it can be seen that the critical spacing $s_{cr,N}$ does not differ from headed anchor theory for all cases where $\psi_{ch,N} = 1$ according to Figure 9 ($h_{ch}/h_{ef} \leq 0.4$). For $h_{ch}/h_{ef} > 0.4$, the ratio $s_{cr,N}/h_{ef}$ is even decreasing. This can be explained by the end distance $x = 25$ mm in all group simulations.

### 5 Design Proposal

Based on the results presented in Section 3 and 4, it is proposed to modify Equations 5 and 7 as follows:

$$\psi_{ch,N} = \left(2 - 2.5\frac{h_{ch}}{h_{ef}}\right) \leq 1.0$$

and

$$s_{cr,N} = 3 \cdot h_{ef}$$

With this approach, no further restriction on the ratio $h_{ch}/h_{ef}$ is needed. The factor $\psi_{ch,N}$ reduces the capacity to “zero” for a theoretical value $h_{ch}/h_{ef} = 0.8$. However, it is recommended to limit the minimum effective embedment depth underneath the channel profile to $h_{ef}^* \geq 40$ mm.

It is noted that with the modifications proposed for Equations 5 and 7, the design approach leads to conservative results for anchor channels with $h_{ch}/h_{ef} > 0.4$ in case of verification of the end anchor since the factor $\psi_{ch,N}$ is derived based on investigations with anchor channels having an end distance $x \geq s_{cr,N}$.

### 6 Conclusion

In this study, the effect of the channel profile on the concrete cone breakout capacity is investigated. Both, tests and numerical simulations have been performed. For comparison, investigations with headed anchors were performed under same conditions. Based on the results described in Sections 3 and 4, the following conclusions can be made:

- For anchor channels with $h_{ch}/h_{ef} \leq 0.4$ no reduction of the concrete cone breakout capacity was found compared to headed anchors. For ratios $h_{ch}/h_{ef} > 0.4$, the results show, that a linear reduction factor well represents the reduction in concrete cone breakout capacity measured in tests and numerical simulations.
A modified factor $\psi_{ch,N}$ is proposed which takes into account the ratio $h_{ch}/h_{ef}$ and not only the effective embedment depth $h_{ef}$ as in current design.

The results described in Section 4 show that the critical spacing is not different for headed anchors and anchor channels. Therefore, it is proposed to also use $s_{cr,N} = 3h_{ef}$ in the design of anchor channels.

The proposed modifications lead to conservative results for anchor channels with $h_{ch}/h_{ef} > 0.4$ in case of verification of the end anchor since the factor $\psi_{ch,N}$ is derived based on investigations with anchor channels having an end distance $x \geq s_{cr,N}$. Further investigations are needed to better utilize the spacing effect of anchor channels with $h_{ch}/h_{ef} > 0.4$.

References:


INFLUENCE OF STATIC LOAD LEVEL ON THE FATIGUE BEHAVIOR OF ANCHOR CHANNELS

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ABSTRACT

Anchor channels used for anchorage of steel components to concrete in fatigue relevant applications such as crane rails, elevators or railway tunnels are commonly subjected to a combination of dynamic actions superimposed on the static loading.

In current qualification guidelines, the fatigue strength for steel failure of cast-in anchor channels is established on the basis of pulsating tension tests with a constant minimum load close to zero, which means without the influence of static loading. In absence of further test results static loads are considered by reduction of the fatigue resistance using a conservatively drawn Goodman-Diagram. With regard to this approach, however, the current state of knowledge is very limited.

In order to investigate the influence of the static load level on the fatigue behavior of anchor channels associated with steel failure, an experimental program comprising high-cycle fatigue tension tests at three different load levels was conducted. The tests were performed on cast-in anchor channels of medium hot-rolled profile size 50/30 made of carbon steel.

This paper illustrates the anchor channel fatigue damage characteristics including the displacement behavior and typical failure mechanism observed during the tests. The evaluation of test results shows no detrimental effect of the static load level on the fatigue resistance. The results obtained for the investigated profile type are compared with similar fatigue tests on anchor channels available in literature.

1 Introduction

In structural engineering, a wide range of various steel components are fastened to the concrete structure by using cast-in anchor channels which consist of a steel profile and a minimum of two anchors connected rigidly to the channel back by forging, welding or screwing. Since anchor channels are installed prior to casting, they provide a robust load-transfer mechanism by means of mechanical interlock between the anchor and the concrete. Special channel bolts placed along the longitudinal axis of the channel profile are used for fixing the attached component.

An increasing amount of anchor channels currently available on the market is frequently used in fatigue relevant application fields. In the case of repeated actions with high load cycles, e.g. resulting
from guide rails of elevators, crane rails, rotating machineries, high speed traffic in railway tunnels or temperature variations in façade elements, a fatigue verification of the anchorage is required to prevent premature fatigue failure of the construction.

In the recent provisions of EN 1992-4\textsuperscript{1}, the fatigue design of fasteners in concrete only applies for post-installed anchors and headed studs. Consequently, the verification of anchor channels subjected to fatigue loading is covered separately by EOTA TR 050\textsuperscript{2}. This Technical Report provides design rules for anchor channels under tension loads which have been qualified for use in fatigue applications. Currently, EAD 330008-02-0601\textsuperscript{3} serves as qualification guideline for anchor channels. In order to obtain the fatigue resistance of a particular product, the guideline specifies assessment procedures by considering all possible failure modes of the whole anchorage system, namely steel failure, concrete cone and pull-out.

Fatigue tension loads on cast-in anchor channels with sufficient edge distance generally experience steel failure\textsuperscript{4,5}. The fatigue strength for steel failure is established through pulsating tension tests with constant minimum load level close to zero. In the absence of further testing, a combination of static and dynamic loads is considered by using the modified Goodman diagram as shown in Figure 1.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{figure1.png}
\caption{Current design concept for combination of static and fatigue loads on anchor channels}
\end{figure}

The fatigue resistance $\Delta F_a$ under pure fatigue loading can be taken from the S-N curve evaluated by tests with $F_{\text{min},a} = 0$, e.g. at $N = 10^6$ load cycles as illustrated above (point a). This value has to be reduced to $\Delta F_b$ depending on the static load corresponding to $F_{\text{min},b}$ (points b). The reduction refers to a straight line between the design values of the fatigue resistance $\Delta F_a$ and the static resistance $F_{\text{stat}}$.

While most of the fatigue data for anchor channels cast into concrete was derived from tests with low minimum load level, research regarding combined static and dynamic loading, which is usually the case in civil engineering structures, is rather limited\textsuperscript{6,7}. In order to investigate the influence of the static load level on the fatigue behavior of anchor channels under tension, fatigue tests on profiles of size 50/30 have been conducted. In this paper, the findings obtained at three different load levels are compared with respect to displacement behavior, typical failure mechanism and number of cycles to failure of the tested specimens. Finally, a discussion of the results compared to tests available in literature is provided.
2 Experimental Investigations

2.1 Test Specimens and Material Properties

The tests were carried out on anchor channels of medium size 50/30 as shown in Figure 2. The specimens consisted of hot-rolled channel profiles and two round headed anchors forged to the back of the channel. The profiles were \( l = 300 \text{ mm} \) long with axial spacing of the anchors \( s = 250 \text{ mm} \) and an excess length of \( x = 25 \text{ mm} \). All channels and anchors were made of carbon steel with hot-dip galvanized coating. Tension tests performed on samples taken from the back of the channel profile resulted in an average yield strength of \( f_y = 429 \text{ MPa} \) and an ultimate strength of \( f_u = 453 \text{ MPa} \). The anchor channels were tested in combination with special hooked channel bolts M16 of quality 4.6.

![Figure 2: Geometry of anchor channels (all dimensions in mm)](image)

The anchor channels were embedded in non-cracked concrete members of low strength C20/25 with sufficiently large member thickness and edge distance. A gap between the end of the channels and the surrounding concrete was provided by using a styrofoam sheet in order to avoid mechanical interlock at this point. The mean compressive strength of concrete determined on standard 150 mm cubes was in between \( f_{cc, 150} = 27.3 - 35.2 \text{ MPa} \) at the time of testing.

2.2 Experimental Program

The experimental program comprised three fatigue test series under pulsating tension loads at different static load levels as illustrated in Figure 3. All tests were carried out with anchor channels loaded via the channel bolt directly over one anchor since this position was identified as the decisive load position leading to the lowest fatigue strength for the investigated profile type.

![Figure 3: Definition of the static load level for fatigue testing](image)
Series 1 was performed at low constant minimum load of $F_{\text{min}} = 1 \text{kN}$ in accordance with the provisions of the current qualification guideline\textsuperscript{3}, which represents testing under pure fatigue loads. Further, to determine the influence of static loads on the fatigue behavior of anchor channels, samples taken from the same production batch were tested in Series 2 and Series 3 at higher constant maximum loads of $F_{\text{max}} = 17 \text{kN}$ and $F_{\text{max}} = 31 \text{kN}$. These two load levels correspond to the design value $F_{\text{Rd, s}}$ and the characteristic value $F_{\text{Rk, s}}$ of the static tensile resistance for steel failure of the tested profile. It can be seen from the static load displacement curve in Figure 3 that Series 1 and Series 2 were performed in the area of linear-elastic load-displacement behavior and Series 3 was already affected by plastic deformations. However, a pronounced yielding point is not apparent.

2.3 Test Setup and Procedure

The tests were conducted using a Schenck Hydropuls hydraulic cylinder with a maximum dynamic load of 80 kN. The realized test setup is shown in Figure 4. The support frame of the cylinder was located in such a way that an unrestricted concrete cone failure could occur. The test rig was placed on steel plates to avoid contact between the support and the anchor channel. The load was applied directly to the channel bolt by means of a threaded coupler. A hinge was provided between cylinder and channel bolt in order to ensure centric load application. Since prestressing forces may influence the fatigue strength of anchor channels, it should be pointed out here that in all tests no installation torque was applied to the channel bolts in order to avoid a superimposition of different effects.

Fatigue testing was carried out load controlled. The test specimens were subjected to cyclic tension loads of constant amplitude according to a sinusoidal wave-form. The cycling frequency was chosen between 2.2 Hz and 12 Hz depending on the specified load range. The applied tension load was measured by means of a calibrated load cell. Two linear variable displacement transducers (LVDT) positioned on either side of the channel bolt were used to measure the displacements of the anchor channel relative to the concrete surface.

![Test Setup Diagram](image-url)
3 Test Results

3.1 Assessment of S-N Curves

In Figure 5, the test results obtained from the fatigue tests at three different static load levels are plotted in the S-N diagram with logarithmic scaling of both axis. Due to the double logarithmic scale, a linear relationship between the load range $\Delta F$ and the number of cycles at failure $N$, ranging between $3 \cdot 10^3$ and $4 \cdot 10^6$ cycles, can be identified.

Consequently, in order to establish the mean fatigue values of the test results, linear regression analysis was carried out. Two regression lines were obtained from the condition that the sum of the squares of the residuals $\log \Delta F$ and $\log N$ respectively is a minimum. The average function of both lines describes the corresponding mean S-N curve for each test series as shown in Figure 5.

The analytical description of this curves representing a straight line is given by the following equations, Eq. 1 for Series 1, Eq. 2 for Series 2 and Eq. 3 for Series 3. Thereby, the two run-out specimens of test series 1 reaching the specified limit number of $5 \cdot 10^6$ and $8 \cdot 10^6$ cycles without failure were not considered in the calculations.

\[
\begin{align*}
\log \Delta F &= -0.239 \log N + 2.34 \quad [kN] \quad k_1 = 4.18 \ ; \ R^2 = 0.97 \quad (1) \\
\log \Delta F &= -0.262 \log N + 2.47 \quad [kN] \quad k_1 = 3.82 \ ; \ R^2 = 0.94 \quad (2) \\
\log \Delta F &= -0.242 \log N + 2.36 \quad [kN] \quad k_1 = 4.13 \ ; \ R^2 = 0.98 \quad (3)
\end{align*}
\]

When comparing the results, it can be seen that the mean S-N curves determined from tests at the above mentioned static load levels lead to very similar fatigue resistance values. The slope of the S-N curves is almost the same. Thus, the damage process for finite fatigue life in case of the tested profile type can be characterized approximately by the constant slope coefficient of $k_1 = 4$. In all test series, correlation coefficients higher than $R^2 > 0.9$ indicate low scatter and good reliability of the fatigue test data.
3.2 Failure Mechanisms

In all the tests steel failure was achieved. Typical failure mechanisms observed from the tests are shown in Figure 6. The cause of failure was governed either by crack propagation in the flanges of the channel profile or failure of the connection between anchor and channel. In some cases also a combination of both failure modes was observed.

In tests with lower static load level the failure occurred mainly by fatigue cracks initiated in the flanges close to the channel lips or the channel back, which represent typical points of stress concentration with notch effect.

With an increase of the static load intensity, a change of the failure mode to failure of the connection between anchor and channel back was detected.

![Image 1) Crack in flange close to channel back](image1)

![Image 2) Failure of connection anchor-channel](image2)

Figure 6: Typical failure modes of fatigue tests
3.3 Displacement Behavior

In order to describe the damage process of anchor channels under pulsating tension at different static load levels, Figure 7 displays the maximum displacements $s_{max}$ versus the number of cycles $N$ on half logarithmic scale for three exemplary fatigue tests of each series performed with identical load ranges. This allows a direct comparison of the results obtained from the three test series.

The curves identify no noticeable effect of static loads on the displacement behavior and the fatigue life. All tests show a very similar displacement development, which is characterized by an almost constant slope to about 80%-90% of number of cycles to failure and followed by a progressive increase of displacements up to failure.

Figure 7: Development of maximum displacements $s_{max}$
4 Discussion

Based on the resulting S-N curves determined at different load levels, which are shown in Figure 8 (left), the Goodman diagram can be created by using the corresponding fatigue resistance values for a certain number of cycles. The obtained results at \( N = 10^6 \) cycles provided in Figure 8 (right). In contrast to the current design provisions\(^2\), where the fatigue resistance has to be reduced linearly according to the static load, the data points 1, 2 and 3 of the S-N curves result in a line parallel to the bisecting line in the Goodman diagram which means an almost identical fatigue strength up to the characteristic value of the static resistance of the tested profile. Above this level a reduced allowable load range may be expected due to high plastic deformations. But this region of inelastic material behavior is not considered for design purposes.

Figure 8: Results of mean S-N curves at \( N = 10^6 \) plotted in Goodman diagram for profile 50/30

In case of channel profiles of size 50/30, the results in Figure 8 demonstrate that design relevant static loads have no noticeable impact on the fatigue resistance of cast-in anchor channels in tension.

Further experimental work with a similar profile type can be found in literature. Güres\(^7\) performed fatigue tests on cast-in anchor channels 38/23 with serrated channel lips as shown in Figure 9 (left) to investigate the effect of different load levels. The tension loads were applied via serrated channel bolts M16 of grade 8.8 located directly over the anchor.
The results in Figure 9 (right) indicate a detrimental effect of increased static loads on the fatigue resistance of the tested anchor channels. In all tests the failure occurred by cracks in the channel profile, which were predominantly located in the profile flanges for tests at $F_{\text{min}} = 1 \text{kN}$. With increasing static load level cracks in the channel back were observed and thus a similar change of failure mechanism was identified compared to the tests with profile 50/30, but without failure of the connection between anchor and channel.

It is worth noticing that all specimens tested by Güres\textsuperscript{7} were prestressed with a high torque moment $T_{\text{inst}} = 120 \text{Nm}$ immediately before the start of the tests. This may have an additional influence on the test results. Particularly in tests with small load ranges at low minimum load level, the remaining high prestressing force can lead to a considerable fatigue life extension by reducing the size of the load range. As a result, the S-N curves become flatter. That means higher slope coefficients are observed compared to tests without installation torque.

5 Conclusion

Anchor channels used as cast-in fastening system in fatigue relevant applications are usually subjected not only to dynamic loading, but to a combination of both dynamic and static actions. Since existing experimental data is mainly limited to fatigue tests at low constant minimum loads, the present research work focuses on the effect of increased static load level on the fatigue behavior of anchor channels.

An experimental program comprising a total of 40 fatigue tension tests have been performed at the Materials Testing Institute University of Stuttgart in order to study the fatigue resistance as well as the displacement behavior and the failure mechanisms of cast-in anchor channels at three different load levels. All tests were carried out on hot-rolled channel profiles 50/30 with two anchors, with pulsating tension loads acting over one anchor.

The results presented in this paper indicate no significant impact of static loads, relevant for fatigue design, on the fatigue resistance for steel failure of anchor channels. Even though the findings are in contrast with the current design provisions for anchor channels according to EOTA TR 050\textsuperscript{2}, the observed behavior is consistent with the fatigue design rules stated in Eurocode 3\textsuperscript{8}, where the influence of static loads on the fatigue strength of steel structures with high notch effect is ignored.

However, it is important to note that the results refer only to the investigated anchor channel profile. Because of different product-specific characteristics due to production method, geometry and material additional work is required to provide more generalized recommendations.

6 Acknowledgement

The research presented in this work was partially supported by the companies JORDAHL GmbH, Nobelstraße 51, 12057 Berlin and HALFEN GmbH, Liebigstraße 14, 40764 Langenfeld, Germany.
References:


6. Hanenkamp, W., Güres, S.: “Zum Tragverhalten von Ankerschienen unter ermüdungsrelevanten Einwirkungen” (Loading behavior of channel anchors under fatigue loads), Stahlbau 73, Heft 9, 668-675, 2004 (in German)


SPECIAL FASTENINGS
PULLOUT STRENGTH OF L-BOLTS

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ABSTRACT

L-shaped anchor bolts (L-bolts) are a time-honored, commonly used anchor to attach steel column base plates, light poles, wood sill plates, and other structural attachments to concrete foundations in the USA. They have lost some popularity with the availability of post-installed anchors. Generally, the anchor is a bent smooth bar with threads on one end and is bent 90 degrees on the other end. L-bolts are inexpensive and easily manufactured out of straight or coiled bars. When used to attach steel columns, there is often little need to consider uplift tension loading on the anchors. The usual loading is a shear load where strength of the anchor depends on how far the anchors are from the free edge of the concrete. However, for sign posts, light poles, and building columns during construction, L-bolts are required to carry tension loading due to overturning moments on the baseplate.

ACI design equations for cast-in-place anchors such as L-bolts capture well the steel tensile and concrete breakout capacities of embedded anchors. However, the design equations for L-bolts are not accurate for the pullout-failure mode. The authors engaged in a testing project to determine L-bolt pullout failure (slip) capacity using a variety of anchor dimensions and concrete strengths.

This paper reviews the behavior of L-bolts and proposes design expressions for tension loading based on published test data.

1 Introduction

L-shaped bar type anchors (L-bolts), a subset of anchors called “hooked bolt” in the ACI Code⁴, have enjoyed extensive popularity in the USA and are still considered a viable anchor type for anchoring steel columns to concrete foundations. However, L-bolts generally have a tensile strength when embedded in concrete less than that of an equivalent welded headed stud anchors or headed bolts.

L-bolts are manufactured from straight or straightened coiled smooth bars and cold bent at the embedded end and threaded on the other end. Steel with yield strengths of 248 MPa to 379 MPa are commonly used for L-bolts. The most often cited modern specification for L-bolt in the USA is ASTM F1554.¹ The availability of the material, ease of bending, and rolling threads on one end plus the ability to upset the threaded end make the L-bolt a very economical cast-in-place anchor alternative.
The American Institute of Steel Construction (AISC) has for many years included sketches of L-bolts in their design manual. The AISC Design Guide, however, only provides design guidance to proportion headed anchor bolts. No design guidance is provided for L-bolts.

2 General Behavior of L-bolts in Tension

In tension, there are four very distinct failure modes for a L-bolt. The four failure modes are: a. steel fracture, b. concrete cone breakout, c. side-face blowout, and d. pullout of the anchor from the concrete. Figure 1 illustrates load-displacement curves for three failure modes a, b, and d as measured in one test series to be discussed later.

![Figure 1: Three modes of L-bolt anchor failure](image)

2.1 Steel failure

A steel failure occurs when the ultimate load that the anchor can carry is controlled by rupture of the steel used in the anchor. This most often happens within the threads.

Average steel failure strength is determined from Equation 1.

\[ N_u = A_{se} (f_{uta}) \]  

(1)

where:

- \( N_u \) = calculated ultimate load [N]

...
A_{sc} = \text{effective cross-sectional area of the anchor} \ [\text{mm}^2] \\
f_{uta} = \text{ultimate tensile strength of the anchor steel} \ [\text{MPa}]

### 2.2 Concrete breakout failure

Concrete breakout failures occur when the embedment depth, $h_{ef}$, of the anchor is shallow. When the failure load magnitude exceeds the tensile strength of the concrete, a semi-conical surface whose apex is at the embedded end of the anchor breaks out. Failure in concrete breakout is brittle. The average concrete breakout equation interpolated from ACI 318-14M\textsuperscript{4} representing cast-in-place anchors in uncracked concrete is given by Equation 2.

$$N_b = 17 \left( f'_{c} \right)^{1/2} (h_{ef})^{3/2}$$

(2)

where:

- $N_b = \text{average expected ultimate load in uncracked concrete} \ [\text{N}]$
- $f'_{c} = \text{concrete cylinder compressive strength} \ [\text{Mpa}]$
- $h_{ef} = \text{effective embedment depth, distance down from surface of concrete to where the tension load is transferred to the concrete} \ [\text{mm}]$

### 2.3 Side-face blowout failure

A side-face blowout failure occurs when the anchor is located near a free edge and a radial bursting force develops in the bearing area of the anchor exceeding the lateral, splitting resistance of the concrete. This failure mode will not be discussed in any more detail other than to mention it is a possible failure mode for an L-bolt anchor when positioned close to an edge of the concrete member.

### 2.4 Anchor pullout failure

The last failure mode is the pullout failure of the L-bolt. This failure mode and its prediction is the primary focus of this article. Pullout occurs when the embedment is deeper than that to form a concrete breakout but not so deep as to provide enough bond and/or bearing capacity to cause rupture of the anchor steel. In Figure 2, several stages of pullout failure of an L-bolt are shown. Initially the load in the anchor is transferred to the concrete by adhesive bond of the anchor shank to the concrete (Figure 2a). As the load increases, but is still less than the steel-rupture load, the adhesive bond breaks down and the load is transferred to the concrete in bearing at the bend (Figure 2b) plus bond of the extended end. In Figure 2c, the concrete is shown being crushed by the high bearing stresses at the bend permitting the typically smooth bar to plastically bend and slip around the 90-degree corner, straightening out as the bar is gradually withdrawn from the concrete by the applied tension load. With continued loading, the L-bolt pulls out of the concrete sometimes causing a small spall at the concrete surface (Figure 2d).
Andrew E.N. Osborn, Donald F. Meinheit and Mark R. Krueger

Figure a - Anchor load transferred by adhesion (bond) to the concrete.

Figure b - Adhesion breaks and load transfers to bearing on the hook. Friction exists along the length of the L-bolt.

Figure c - Concrete crushing starts at the hook bend.

Figure d - Sufficient crushing of the concrete at the bend allows the hook to straighten out and the anchor shank pulls out of the concrete, usually accompanied by a small popout.

Figure 2: Stages of a pullout failure
3 Prediction Models for Pullout Failures

3.1 WJE 1993 tests

In 1993, Wiss, Janney, Elstner Associates, Inc. (WJE) conducted a series of 35 tension tests on L-bolts embedded in concrete as part of an in-house research project. Results for these tests and others found in literature were analyzed. Listed in Reference 8 is the database of L-bolt anchor tests assembled by WJE. Observations from the 1993 WJE tests showed the following features:

- Concrete breakout failures appear to be proportional to the embedment depth instead of the embedment depth squared. This observation was also verified by simplified finite element analysis. Concrete breakout failure occurred only for shallow embedment.

- A minimum leg extension, \( e_h \), of 6 \( d_a \) was estimated as being necessary to develop the tensile strength of the steel, assuming the embedment depth is greater than that which would cause a concrete breakout. The standard leg extension typically supplied with L-bolts is about 4.5 \( d_a \).

- A minimum embedment depth needed to force a pullout or steel failure depends on \( e_h \), the surface roughness condition of the anchor, and \( f'_c \). However, based on the equations presented in the remaining part of the paper, if the leg extension, \( e_h \), exceeds about 6\( d_a \), a pullout failure can be prevented.

The WJE research project resulted in a recommended design equation similar in form to the recommended design equation presented in Reference 6, both of which are presented in Reference 8. Also listed in Reference 8 are all published and unpublished sources of test data known to the authors. A significant number of the tests were conducted on L-bolts embedded in grouted cells of concrete masonry units. In total, the compiled database contains the results of 97 tests, 60 of which are pullout failures. The ranges of test variables in the database were as follows:

<table>
<thead>
<tr>
<th>Test variables</th>
<th>Range in database</th>
<th>Range for pullout</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (( f'_c ))</td>
<td>13.6 - 45.2</td>
<td>13.8 - 34.5</td>
</tr>
<tr>
<td>Anchor diameter (( d_a ), (mm))</td>
<td>9.5 - 51</td>
<td>9.5 - 25</td>
</tr>
<tr>
<td>Embedment depth (( h_{ef} ), (mm))</td>
<td>51 - 508</td>
<td>76 - 343</td>
</tr>
<tr>
<td>Hook extension (( e_h ), (mm))</td>
<td>28.5 - 149</td>
<td>28.5 - 114</td>
</tr>
<tr>
<td>Embedment depth ratio (( h_{ef} / d_a ))</td>
<td>3.5 - 24</td>
<td>6.0 - 19.8</td>
</tr>
<tr>
<td>Hook extension ratio (( e_h / d_a ))</td>
<td>1.6 - 8.6</td>
<td>2.5 - 4.6</td>
</tr>
</tbody>
</table>

3.2 ACI 318 code pullout equation

The current ACI design equation for hooked-bolt anchors makes reference to a report by Kuhn and Shaikh. The data presented in Reference 6 is essentially the same as the database assembled by WJE in Reference 5. In Reference 6, the average ultimate strength of a L-bolt anchor was proposed as the sum of a bond component and a bearing component, as shown in Equation 3. The coefficients in this equation are based on the Kuhn and Shaikh analysis of test results in the database for L-bolt anchors.
The Kuhn and Shaikh predicted failure loads in uncracked concrete, by Equation 3, are compared to the pullout test data in Figure 3.

\[
N_{p, \text{average uncracked}} = 1.5 f'_c \epsilon_h d_a + 2.1 \pi (h_{ef}) (d_a) \quad [N]
\]

where

- \(f'_c\) = concrete cylinder compressive strength, (MPa)
- \(\epsilon_h\) = distance from the inner surface of the L-bolt shaft to the outer tip of the L-bolt [leg extension], (mm)
- \(d_a\) = outside diameter of L-bolt, (mm)
- \(h_{ef}\) = effective embedment depth, distance down from surface of concrete to where the tension load is transferred to the concrete, (mm)

Geometric definitions of \(\epsilon_h\), \(d_a\), and \(h_{ef}\) are shown in Figure 2a.

![Figure 3: Pullout strength prediction equation from Reference 6 compared to the database. Note also the comparison to the ACI 318-14 equation.](image)

The design equation in ACI 318-14 Chapter 17, is a condition representing a L-bolt embedded in cracked concrete. All design equations in Ch 17 are also design capacity values that are some percentage less than the average value, following the 5% fractile approach. The amount less than the average depends upon the standard deviation of the test data around the mean and the number of tests. Because the bearing part of Eq. 3 cannot take effect unless the bond strength is exceeded, one
approach is to ignore the second term of Eq. 3. Assuming the second term is ignored can result in a characteristic design equation for the bearing strength of about 75 percent of the average uncracked value. Hence, if the coefficient of variation of the data is about 15 percent, the 1.5 factor reduces to 1.25. Multiplying by the reciprocal of the typical factor to account for concrete cracking, that is, 1/1.4, causes the 1.25 factor to become 0.89, resulting in a close approximation of the ACI 318-14, Equation 17.4.3.5 as presented in equation 4.

\[ N_{p, \text{design, cracked}} = 0.9 f'_c \, e_h \, d_a \, [N] \]  \hspace{1cm} (4)

If in Figure 3 the bond portion of the prediction equation is ignored, design based only on bearing is obviously conservative.

### 3.3 Re-evaluation of test results and proposed new equation

Although the ACI 318 design equation is a bearing equation, it is appropriate to ask whether the data on L-bolt pullout failures is predominated by bearing, bond, or can be predicted by some combination of both behaviors. Further review of the test results listed in Reference 8 seems appropriate considering the conservative nature of the design equation in ACI 318-14 as reflected in Figure 3. There are 97 L-bolt test results listed in Reference 8, of which 60 are pullout failures. However, five of the pullout failures were for L-bolts with teflon wrapped around the shank of the anchor when it was cast into the concrete. The objective of the tests with the teflon wrap was to eliminate the bond and test only the bearing of the hook on the concrete. Review of the load-deflection plots for these five tests showed that the addition of the teflon wrap significantly affected the overall behavior and reduced the ultimate load. These tests were considered biased and eliminated from the pullout test group but they do illustrate the contribution of bond to the total pullout capacity. The range of the variables in the pullout group appears sufficiently broad to qualify as unbiased toward any specific geometric or material condition. A statistical significance test was performed on the ratio of the test result to calculated result to see if the grout (from Reference 7) and concrete materials belonged to the same population. The average grout strength was corrected using the square root of the two strengths to adjust the average grout strength to the average concrete strength. The hypothesis that the two groups of data are in the same population was verified, and therefore, the grout tests are included with the concrete tests in the analysis. Our analysis showed that bond affects bearing and bearing affects bond such that ACI’s reliance on the first part of Eq. 3 only was overly conservative. Instead, a pseudo-bearing only equation that includes the influence of bond, or an equation that includes the effects of bearing and bond, is more appropriate.

**Possible prediction models:** The assumption in Eq. 3 is that the pullout strength is the additive contribution of the bond strength of the anchor shank and extension to the concrete plus the bearing strength of the L-bolt leg extension. This approach has been taken in the past for hooked reinforcing bars but was abandoned in the early 1980s.

Data was analyzed to examine whether the pullout strength is dominated by bond or bearing. Capacity formulas based solely on bond or bearing area, when considered separately, offer no distinctly better correlation of the measured test results to one set of variables over the other, although the bearing data does almost statistically go through a zero origin.
Several different models were subsequently investigated. Prediction models of the format shown below in Eq. 9 were considered.

\[ N_p = C (d_a)^\alpha (h_{ef})^\beta (f_c^\prime)^\gamma \delta \]  

(5)

where:

- \( C \) = linear correlation coefficient
- \( \alpha \) = exponent on the shank diameter \((d_a)\) of the anchor
- \( \beta \) = exponent on the hook leg extension \((h_{ef})\)
- \( \gamma \) = exponent on the effective embedment depth \((h_{ef})\) of the anchor
- \( \delta \) = exponent on the compressive strength \((f_c^\prime)\) of concrete

An alternate approach is to assume an additive model as was done by Kuhn and Shaikh and in Reference 6 and WJE in Reference 3. The form in Eq. 6 was used for a least squares fit of the data.

\[ N_p = C_1 + C_2 (f_c^\prime d_a h_{ef}) + C_3 (f_c^\prime \pi d_a h_{ef}) \]  

(6)

where:

- \( C_1 \) = linear regression intercept
- \( C_2 \) = correlation coefficient on bearing stress component
- \( C_3 \) = correlation coefficient on bond stress component

Multiple least squares, linear regression analyses, with multiple independent variables, were made using the format in Eq. 5 and Eq. 6. In addition, versions of these equations were analyzed with only bearing considered or with only bond considered. Regression equations were developed with coefficients that numerically give the best fit to the data. However, for design, simpler equations and rounded coefficients are desirable. Additionally, the characteristic design equation is taken as a percentage of the equation representing the average strength and considers the coefficient of variation. Each of the trial equations were simplified, exponents rounded, and the 5 percent fractile value determined. Hence there were four possible equations presented and discussed in Reference 8.

In the final analysis, the authors determined that an additive bond plus bearing equation was undesirable because the tendency would be to simply cut off the bond term, resulting in the same overly conservative formulas currently in the ACI code. A bond or friction only model is unlikely to be accepted since bond and friction are not considered to be a reliable method of assuring load transfer. Instead, our recommendation is to use one of two approaches, a bearing only formula (Equation 7) or a bond/bearing formula (Equation 8). These two equations are presented below. Both represent expected average pullout strengths in uncracked concrete based on the database of 55 pullout tests.

\[ N_p, \text{average bearing uncracked} = 65.5 \ d_a \ h_{ef} \ [N] \]  

(7)

\[ N_p, \text{average (bond/bearing) uncracked} = 4.33 \ d_a^{3/4} \ h_{ef}^{1/2} \ f_c^\prime 1/8 \ [N] \]  

(8)
where the variables are $f_c$ [MPa], $d_a$ [mm], $e_h$ [mm], and $h_{ef}$ [mm], identified previously. Both are reasonable predictors of the average pullout capacity and much more accurate than the current ACI 318-14 equations.

The compressive strength of the embedding material ranges from 13.8 Mpa to 34.5 MPa. The analysis of the data strongly indicates that the compressive strength of the embedding material is not a significant variable. Therefore, extrapolation to compressive strengths greater than the range tested should be conservative.

A comparison of the measured uncracked test load to the predicted uncracked test load by Eq. 7 and Eq. 8 are shown plotted in Figures 4 and 5, respectively. The existing uncracked ACI code strength prediction, Eq. 4, is seen to be about half the actual measured strength is also shown for comparison. Therefore, an increase in the pullout capacity design for L-bolts seems warranted.

Figure 4: Comparison of the predicted and measured pullout capacities by ACI and equation 7
Figure 5: Comparison of the predicted and measured pullout capacities by ACI and equation 8

It is recommended that a design equation of the form in Eq. 9 or Eq. 10 be adopted, both of which are based on the 5% fractile of the data. In addition, it is recommended that the leg extension requirements in the ACI 318 code remain and that the minimum embedment depth for all L-bolt anchors be 15d_a if their design is by Eq. 9. Since embedment depth is a variable in Eq. 10, the same restriction does not apply. In no case should the value of the predicted strength be taken larger than the steel strength, the concrete breakout strength, or the side blowout strength. If there exists the possibility of concrete cracking at the anchor, the design values should be divided by 1.4.

$$N_{p, 5\% \text{ fractile design uncracked}} = 37 \, d_a \, e_h \, [\text{N}] \quad \text{(Eq. 9)}$$

$$N_{p, 5\% \text{ fractile design uncracked}} = 2.7 \, d_a^{3/4} \, e_h \, h_{ef}^{1/2} \, f_c^{1/8} \, [\text{N}] \quad \text{(Eq. 10)}$$

where the variables are $f_c$ [MPa], $d_a$ [mm], $e_h$ [mm], and $h_{ef}$ [mm] as defined previously.

4 Summary

Testing data has existed on the pullout strength of L-bolt anchors for many years but the data had not been thoroughly analyzed and published until Reference 8 was published in 2014. Results reported in this paper indicate that the pullout failure design strength for L-bolt anchors could be increased and still be consistent with the provisions in the ACI code that allow design models to be in substantial
agreement with comprehensive tests. The current ACI equations were made by simply removing the bond term in Reference 6. Instead, the authors have found that bond influences the bearing of L-bolts and therefore, simply truncating the Reference 6 formula is unnecessarily conservative. The authors’ believe the results presented herein are comprehensive and meet the requirements of the code provisions. Equations representing the pullout strength of L-bolt anchors correlate well with the test data. The existing pullout failure load model for a L-bolt anchor in ACI is very conservative and can be increased by a factor of at least two.

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CONCRETE SCREWS AS POST INSTALLED REINFORCEMENT

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ABSTRACT

In the last two decades concrete screws have gained more importance as anchoring system because of their easy installation and high load bearing capacity. Motivated by this growing importance adhesive screw anchors with diameters up to 22 mm have been developed, which can be additionally installed with bond resins. Because of their ability to transmit high loads over short bond length screw anchors may also be used as post installed reinforcement, especially as shear reinforcement.

Due to growing traffic volume and more restrictive design standards, structural engineers have a requirement for post installed reinforcement to strengthen existing concrete structures. Especially the amount of calculated shear reinforcement has increased in connection with the introduction of the Eurocode standards. Based on these considerations and taking into account the fact that the age of infrastructural constructions such as bridges is also rising, methods to increase the shear strength of existing structures are needed.

Unfortunately, current structural solutions that aim to enhance the shear capacity suffer from many technical and economic problems. Therefore the unit of Concrete Structures and Bridge Design of the University of Innsbruck is developing a new shear strengthening method using post installed concrete screws. The advantages of this system are the easy installation which comes along with screw anchors and can be done from the lower surface of the structure without any disturbance of the traffic and the efficiency of the screws concerning the growth of the shear load capacity.

The new method is analyzed in several series of shear tests with beams with a four point bending configuration. In several different variations of screw diameters and installation geometry all tests resulted in a remarkable increase in load capacity using concrete screws as post installed shear reinforcement. Three tests were made with dynamic loads of 1.25 million and 5 million load cycles. The results showed that this system allows a significant increase in shear strength of concrete structures under dynamic load conditions such as bridges in a very robust and economic efficient way.

1 Introduction

A large number of concrete bridges in Central Europe were built from the 1960s up to the 1980s (1,2) according to design codes of the 1950s and 1960s.
In the last decades traffic volume and traffic weight have been increasing remarkably and will increase furthermore according to current studies. This is also considered in the actual load approaches in codes such as Eurocode 1 with significant higher loads to be applied in actual structural analysis, as shown in. The application of these load approaches to aged bridges combined with new design and construction rules for concrete bridges leads to a lack of existing reinforcement in these structures. As shown in especially the maximum shear load capacity without shear reinforcement calculated on basis of Eurocode 2 is up to 50% smaller, compared to Austrian Standard B4200 which dates from 1957. On the other hand, the necessary stirrup reinforcement for shear and torsional loads calculated on basis of Eurocode 1 is up to 20% higher compared to older standards.

Figure 1: Attached steel plates at bridge girders and drilled through bars as examples for current shear strengthening methods, taken from (left) and (right)

Figure 1 shows two examples of shear strengthening systems which are currently in use. The left figure shows steel-plates that are glued to the bridge girders and attached threaded bars, which are drilled through the bridge deck to ensure a confinement of the bending compression and tension zone. Another widely used system are drilled through threaded bars as shown in the right side of Figure 1. For the installation of both examples the pavement and the sealing of the bridge has to be removed and therefore at least one lane of the bridge must be closed. On the other hand systems with glued anchors that are installed from underneath the bridge lose their strength at temperatures higher than 80°C according to Eligehausen.

Although several guidelines such as the “German Structural Assessment Provisions for Older Road Bridges” allow modified design approaches to assess the shear resistance of existing concrete bridges, based on practical experiences a need for new strengthening systems is given. Especially for shear strengthening, new methods are demanded which are cost-efficient, simple to install and based on robust force transfer mechanisms such as interlock. Since a total lock down of bridges especially for highways results in large economic losses a new strengthening system should be installed without disruption of the ongoing use.

Concrete screws as post-installed shear reinforcement can be installed from the lower surface of concrete structures since the load transfer is based on interlock between screw thread and drilled hole.
in the concrete structure as shown in Figure 2. Therefore this paper investigates the use of these shear strengthening elements for concrete structures with dynamic loads such as bridges.

2 Description of the post installed concrete screws

Concrete screws or screw anchors are normally used as a fastening system in cracked or non-cracked concrete. They are installed in hammer-drilled holes. The diameter of the drilled hole is given by the screw-manufacturer. At the tip of the screw a thread is provided, as shown in Figure 2 and Figure 3, larger in diameter than the drilled hole. The screws are driven in with a special impact driver and thus cut threads into the concrete surface of the drill hole as shown in the right image of Figure 2. The concrete screws used have special welded marks at the cutting thread to ensure a correct interlocking connection between concrete and anchor.

For the shear tests three different types of screws provided by the Manufacturer TOGE Dübel were used. These screws are built with a standard ISO thread at the rear end for fixing attachments with standard washers and nuts. The form-locked connection on the outside of the test-beams was accomplished with nuts, Nord-Lock elements and washers as shown in Figure 3. Additionally to the interlocking connection of the screws a two-component vinylester resin can be used to achieve a bond connection between anchor and concrete at the whole length of the screw. To ensure the correct bond properties drills with small manufacturing tolerances have to be used and the drill holes need to be cleaned by flushing and brushing several times.
The TOGE TSM-16 screw as shown in Figure 3 needs a borehole diameter of 16 mm for a correct installation and is equipped with a standard M16 ISO-thread. This screw with the shown length of 340 mm was especially developed for the shear tests. The TSM-22 screws, which need a borehole diameter of 22 mm are standardised products and are used to fasten noise barriers on railway bridges for example (see 11).

![Figure 3: Used types of concrete screws as post-installed shear reinforcement for the experimental research with nominal diameters of 22 mm and 16 mm](image)

3 Experimental research

To determine the behaviour of concrete screws as post-installed shear reinforcement, 32 test specimen in three test series were investigated at the University of Innsbruck. In each test series several different parameters were investigated. The aim of the first test series was to show the suitability of concrete screws as shear strengthening elements. Therefore the TSM-22 screw was used with and without mortar, and the installation direction was varied from the underside and from the top of the girder. These tests were performed with six and eight strengthening elements in the shear zone. On basis of the test results of the first series screws with smaller diameters (TSM-16) were investigated in the second series along to TSM-22 in different geometrical arrangements, as shown in Figure 4. To obtain smaller crack widths of the critical shear crack torque was applied at the nuts of several screws to generate axial prestress in the screws. The last test series was performed to determine the influence of the installation depth of the screws. One test of the first series and two tests of the second test series were performed with cyclic loads to investigate the dynamic behaviour of screw anchors as shear reinforcement. These test were precracked with a load of 160 kN. Afterwards 1.25 and 5 million load cycles between 70 kN and 140 kN were performed respectively. Because none of the three tests failed during cyclic loading, the ultimate load was determined by static loading afterwards. In this paper an overview of the new system will be given and several tests will be discussed in detail. Further information to all tests with static and dynamic loads can be found in 12–14.
3.1 Dimension of specimen

All tests were performed as four-point-bending tests with girders with rectangular cross-section. Figure 5 shows the geometrical dimensions and the used reinforcement for all test beams. As can be seen the total length of the specimen was 350 cm with a distance of 250 cm between the supporting points and 50 cm between the load application points. The cross-section width was 22 cm and the height 32 cm. This results in a shear span to effective depth ratio a/d of 3.45, which is similar to the test series EA performed by Leonhardt and Walther\textsuperscript{15}.

As shown in Figure 5 all test beams were reinforced with four bars of 20 mm in diameter as flexural reinforcement to avoid early bending failure. Two reinforcing bars were additionally placed in the compression zone. At the bearing points and between the points of load introduction stirrups with a bar diameter of 10 mm were installed. On this basis on both sides of the beam 80 cm could be used to investigate the shown concrete screws as post-installed shear reinforcement with several different installation parameters and geometrical arrangements of the screws.
3.2 Material properties

For both test series ready-mixed concrete of the strength-class C25/30 F45 GK16 XC1 according to the European standard EN206-1 was used. Parallel to the shear tests the compressive and tensile strength of the used concrete was determined using cubes with a side length of 150 mm, which were stored under the same conditions as the test beams.

It should be mentioned that the measured compressive strength of the second test series differs considerably from that of the first and third test series. On average the values deviate by 7.40 N/mm² and 11.8 N/mm² respectively, as can be seen in Table 1. A significant subsequent hardening could not be measured during the test periods.

Table 1: Concrete compressive and indirect tensile strength measured upon cubes with 150 mm side length and Young’s Modulus of used concrete

<table>
<thead>
<tr>
<th>test series</th>
<th>compressive strength $f_{c,cube}$ [N/mm²]</th>
<th>tensile strength $f_{c,sp}$ [N/mm²]</th>
<th>Youngs modulus $E_c$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>35.42</td>
<td>2.23</td>
<td>29 090</td>
</tr>
<tr>
<td>2</td>
<td>28.03</td>
<td>2.40</td>
<td>25 697</td>
</tr>
<tr>
<td>3</td>
<td>39.88</td>
<td>3.20</td>
<td>28 330</td>
</tr>
</tbody>
</table>

For all three types of used shear strengthening elements uniaxial tension tests were performed. With three tests of each type a mean stress-strain behaviour was determined. The tensile test diagram of the TSM-22 M24 anchor shows a highly ductile steel with an elongation after fracture of over 16 % and a maximum stress of about 650 N/mm². In comparison, the screw anchors TSM-22 M20 and TSM-16 M16 show a higher maximum stress over 700 N/mm² but a smaller elongation after fracture. The yield stress of all screws is defined as 0.2% proof stress and was determined with about 520 N/mm² for all used screws.

3.3 Procedure of shear tests

Figure 6 shows the test set-up of the shear tests. The load was applied with a computer controlled hydraulic jack with a maximum load of 630 kN. The tests were performed path-controlled with a displacement of 1 mm/min. At steps of 30, 60, 120, 200 and 250 kN the tests were stopped to manually measure the crack development and crackwidth.

The beam deformation was recorded with six inductive displacement transducers, three on each side of the beam. Two transducers were attached in the middle of the beam while the four others were applied right at the points of load application as shown in Figure 6.

To quantify the acting forces in the reinforcement elements all concrete screws of the first test series were applied with two strain gauges of type HBM LY41-6/120. In order to prevent any damage to the strain gauges and cables, the whole screw shaft was wrapped with electrical tape with the exception of those screws installed with injection mortar. In these screw anchors the cables were attached in longitudinal millings to ensure a correct bond behaviour over the full screw length. In the third test series force-measuring rings between the concrete surface and washers were used to determine the screw forces instead of strain gauges, as shown in Figure 6 too.
In the second and third test series a digital image correlation system was used to record the development and opening of the shear cracks on both sides of the test beams continuously. This system facilitates taking pictures with two high speed cameras of both shear areas of the beam every second and simultaneously records the displacement and force of the hydraulic jack as well. These photos are analysed after the tests with special software to obtain the information about crack width and propagation on the surfaces of the specimen. The used cameras, each with two lights to ensure a good illumination of the surface of the beam, are shown in Figure 6.

![Figure 6: test setup and used systems of measurement](image)

### 3.4 Test results

The following figures show some of the parameters investigated in the three performed test series. In the first test series two test were performed with 6 TSM-22 screws installed from the upper surface of test beams to determine the influence of the anchorage of the screws in the flexural tension or in the compression zone.

Figure 7 shows the force-deflection curves of the tests with 6 and 8 reinforcing elements with a nominal diameter of 22 mm installed without bond resin. The beam deflection is determined as mean value of the four displacement transducers attached under the points of load application (see Figure 6). It can be seen in Figure 7 that a significant load increase was obtained with 6 and 8 TSM-22 in unglued installation compared to the reference test without shear reinforcement. The influence of the installation direction in the maximum load is negligibly small with 5 % and 8 % for 6 screws as well as 8 screws. The beam deflection is higher for the tests with screws installed from the upper side because of the higher crack width of the bending cracks in the anchorage zone of the concrete screws.

Figure 8 shows the influence of screws installed with bond mortar compared to tests without for two different types of screws (TSM-22 and TSM-16). The amount of shear reinforcement is 15.21 cm²/m for TSM-22 and 16.75 cm²/m for TSM-16. This means a variation of the shear reinforcement ratio \( \rho_{sw} \) by 0.07 %. All tests show significant higher failure loads compared to the reference tests. The both tests with glued screws in Figure 8 reached up to 60 % higher failure loads compared to the
Jürgen Feix and Johannes Lechner

Unglued tests for TSM-22 and up to 30% higher loads for TSM-16. It can be seen, that the tests with screws in glued installation have a higher component stiffness because of smaller crack width of the critical shear cracks. In Figure 8 a significant loss of stiffness is shown for the test with 6 unglued TSM-22 screws at a load level of around 150 kN. This is in the range of the shear load capacity of an unreinforced test beam.

In the third test series the influence of the installation depth was investigated. Three depths of the drilled hole were used. 290 mm with the top of the screw at the top level of the upper flexural reinforcement bars and 260 mm with the top of the screws at the lower level of the upper flexural reinforcement bars. Additionally 230 mm with the anchorage of the screws significantly under the flexural compression zone of 7.3 cm at maximum load, was investigated. A significant influence of...

Figure 7: Force-Displacement diagrams of shear tests with installation of concrete screws from the upper side and the lower side of the beam respectively

Figure 8: Force-Displacement diagrams of shear tests with and without resin of TSM-22 and TSM-16 concrete screws
the installation depth can be seen in Figure 9. All tests show higher failure loads compared to the reference test. The load increase is 46% for the top of the screw at the upper level of the upper reinforcement bars and two times higher compared with the tests with smaller installation depths. The difference between 260 mm and 230 mm installation depth is with 5% noticeably smaller.

Figure 10 shows the test results of the performed cyclic loaded tests compared to the static loaded reference tests with the same test setup. In the first test series one test reinforced with 6 TSM-22 in unglued installation was loaded by 1.25 million load cycles between 70 and 140 kN after precracking. The test did not fail during cyclic loading and therefore was loaded until failure statically after the dynamic load. Figure 10 shows a by 20% higher failure load of the dynamic test.
and a permanent deformation of about 2 mm out of the dynamic load. The beam deflection at failure is nearly equal compared to the static test. Therefore it can be seen that the stiffness of the test beam was significantly higher after dynamic loading.

In the second test series two tests with glued screws were loaded with 5 million load cycles at same load level. None of these two tests failed due to cycling loading. The right picture of Figure 9 shows for both tests higher failure loads compared to the static tests. The permanent beam deflection resulting out of dynamic load is also smaller compared to the test of the first test series although the number of load cycles was 4 times higher. During the tests the largest growth of beam deflection and crack width was measured for the first 1.2 million load cycles. The beam deflection grew nearly linearly afterwards.

4 Conclusion

The paper presents a new system for shear strengthening of concrete structures with a lack of shear reinforcement. The new idea is to use concrete screws, known as anchoring elements. These elements have the big advantage of a very simple and fast installation and the robust force transfer mechanism based on undercut. Because of this, the system can be installed from underneath the structure and can be loaded immediately. Therefore structures like bridges can be strengthened without interruption of their usage such as closure of single lanes or of the whole bridge.

In over 30 experimental test with concrete beams the concrete screws showed their excellent suitability as post-installed reinforcing elements. In all tests a higher failure load was achieved compared to reference beams without transverse reinforcement. The used concrete screws can be used with additional bond resin, which means an additional corrosion protection too. Therefore several tests with this bond resin were performed and compared to their equal tests without resin. It was observed that the tests with glued concrete screws have higher failure loads and smaller crack widths. A test series with different installation depths of the concrete screws showed a significant influence of the screw anchoring underneath or at the upper level of the upper flexural reinforcement bars for beams with rectangular crosssection.

The tests showed, that the new system can be used as shear reinforcement system successfully. In the meantime it has already been used in several pilot projects. Before the usage as standard construction method several additional tests should be done to determine the influence of parameters like the horizontal distance of the elements or the prestressing of the screws by torque.

5 Acknowledgement

We want to thank our supporters, especially TOGE Dübel GmbH & Co. KG from Nürnberg, Germany for the excellent and ongoing collaboration and the Deutsche Bahn Netz AG for the financial support of our tests.

References:


BEHAVIOR AND DESIGN OF ANCHORAGES WITH SHEAR LUGS

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ABSTRACT

Anchorages with shear lugs are often used in connections to concrete yet there are currently no design provisions for these in either the fib Bulletin 58 “Design of Anchorages in Concrete”\textsuperscript{1} or in the American Concrete Institute ACI 318-14 “Building Code Requirements for Structural Concrete and Commentary.”\textsuperscript{2} This paper provides a behavioral model and design guidelines for anchorages with shear lugs that is based on tests performed by Rotz and Reifschneider\textsuperscript{3,4} and by Michler\textsuperscript{5}. The paper also provides references to more detailed design procedures developed by Michler\textsuperscript{5}. The model presented provides a method for determining the shear bearing strength of anchorages with shear lugs with or without applied axial loads. The method uses a basic shear bearing strength for anchorages with no axial load then reduces this value for cases with applied axial tension while increasing the value for cases with applied axial compression. The model requires that the bearing force on the shear lug be included in determining the tension in the anchor from moment. The model has been accepted by the fib Task Group 2.9 for the next edition of the fib Bulletin 58 “Design of Anchorages in Concrete” and is in the ballot process for incorporation into American Concrete Institute ACI 318 “Building Code Requirements for Structural Concrete and Commentary” and ACI 349 “Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary.”

1 Introduction

This paper presents a behavioral model and design guidelines to be used for connections utilizing shear lugs (Fig. 1). The behavioral model is based on tests performed by Rotz and Reifschneider\textsuperscript{3,4} and by Michler\textsuperscript{5}. The model is based on determining a basic bearing strength in shear on the embedded portions of the connection in the absence of applied axial load and then adjusting this bearing strength to accommodate the effects of applied axial tension or applied axial compression.

The concrete related failure modes of connections with shear lugs include concrete breakout of the anchors in tension, bearing failure in shear, and concrete breakout in shear. Concrete breakout of the anchors in tension is no different than for other types of connections with anchors and must be considered in accordance with the applicable design standard. Steel related failure modes include anchor tension, anchor shear, and steel failure of the shear lug and must be taken into consideration. The design of the steel and welds for the shear lug should be carried out in accordance with applicable standards. As discussed in the background section, the steel failure of the anchors in shear...
is a secondary failure. This paper addresses the following concerns that need consideration with shear lug connections:

1. Bearing strength in shear
   a. basic bearing strength
   b. modification factor for axial tension
   c. modification factor for axial compression
   d. effective bearing depth of shear lugs
   e. anchor tension due to eccentricity of the bearing reaction on the shear lug

2. Concrete breakout strength in shear

![Anchorage with shear lug](image)

**Figure 1: Anchorage with shear lug**

### 2 Background

Rotz and Reifschneider\(^3,4\) performed a series of tests using flush mounted base plates with and without shear lugs, and embedded baseplates with and without shear lugs. Figure 2 provides an overview of the types of tests. Figure 3 provides the shear load versus displacement graph for the tests run with no applied axial load from the Rotz and Reifschneider\(^3\) report. As shown in Figure 3, two distinct failure modes were observed. The first was bearing failure at a high load and small displacement followed by steel failure of the anchors at a lower load and much larger displacement. Figure 4 shows the conditions just prior to both failure modes. As with the concrete breakout failure mode, a crack begins to form and propagate until a critical length is reached then an abrupt failure occurs. It should be noted that the steel failure of the anchors in shear is a secondary failure and not the subject of this paper.

Michler\(^5\) undertook a substantial experimental and analytical program on anchorages with shear lugs. In his work, Michler developed four different models of different complexity. Model A is based on a simple fitting to test results with the shear lug and base plate embedded or not. Model B is a comprehensive iterative model with compatibility of deformations and is recommended for a more detailed analysis than provided in this paper. Model C was also iterative and a simplification of Model B. Model D is based on rigid behavior and considers anchor location and external loading. This model also provides a simple equation for determining the effective depth of shear lugs.
Regarding concrete breakout in shear, very little testing has been done in this area. Two large scale shear lug tests exhibiting concrete breakout were performed by Gomez, et al\textsuperscript{6} and are discussed in the section on concrete breakout strength below.

Figure 2: Test details from Rotz and Reifschneider\textsuperscript{3}

Figure 3: Shear load versus horizontal displacement from Rotz and Reifschneider\textsuperscript{3}

Figure 4: Bearing failure mode and steel failure mode for surface mounted plate with shear lug
3 Bearing Strength in Shear

This section covers the derivation and verification of the basic bearing strength and how to account for the effects of applied axial load on the basic bearing strength. It also addresses the effective bearing length of both shear lugs and anchors as well as the possibility of anchor tension resulting from the eccentricity of the bearing reaction on the shear lug.

Figure 5 provides a graph of the Rotz and Reifschneider\textsuperscript{3} test results showing the linear trend of the bearing failure in shear with both applied tension and compression. In Figure 5, the vertical axis is the applied axial load while the horizontal axis is the bearing failure load divided by the effective bearing area of the connection and the concrete compressive strength. As discussed in Section 3.4 the effective bearing length is twice the thickness of the member. For these tests, this simplification only applies to the effective bearing area of the anchors and that of the shear lug in test series DU. The anchors used were 20.3mm x 25.4mm bars with the 20.3mm in the direction of the shear load.

Table 1 provides a full summary of the Rotz and Reifschneider\textsuperscript{3} test results. In addition to the information in Table 1, the ultimate tensile stress, $f_{\text{ult}}$, of the anchors was 464 Mpa.

Table 1: Summary of Rotz and Reifschneider\textsuperscript{3} test results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Load (kN)</th>
<th>Displacement (mm)</th>
<th>Effective Bearing Area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Shear</td>
<td>Axial</td>
<td>Shear</td>
</tr>
<tr>
<td>AD1</td>
<td>33.32</td>
<td>1030</td>
<td>0</td>
</tr>
<tr>
<td>AD2</td>
<td>35.26</td>
<td>822</td>
<td>150</td>
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<tr>
<td>ADFT1</td>
<td>31.20</td>
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<td>31.11</td>
<td>765</td>
<td>133</td>
</tr>
<tr>
<td>ADFT3</td>
<td>32.54</td>
<td>465</td>
<td>209</td>
</tr>
<tr>
<td>ADFT4</td>
<td>32.75</td>
<td>1017</td>
<td>342</td>
</tr>
<tr>
<td>BU1</td>
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<tr>
<td>BU4</td>
<td>32.22</td>
<td>234</td>
<td>338</td>
</tr>
<tr>
<td>BUC1</td>
<td>32.32</td>
<td>1833</td>
<td>-431</td>
</tr>
<tr>
<td>BUC2</td>
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</tr>
<tr>
<td>CUF2</td>
<td>36.33</td>
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</tr>
<tr>
<td>CUF3</td>
<td>36.71</td>
<td>447</td>
<td>565</td>
</tr>
</tbody>
</table>

Figure 5: Linear trend of bearing failure in shear with axial load
Based on the linear trend shown in Figure 5, Eqs. (1), (2), and (3) provide a general linear model for predicting the bearing capacity in shear.

Basic bearing strength in shear: \[ V_{\text{beg,0}} = k_0 f'_c A_{\text{beg}} \] (1)

Bearing strength with applied axial tension: \[ V_{\text{beg,N}} = k_0 f'_c A_{\text{beg}} \left( 1 - \frac{N}{\sum A_i f_{ui}} \right) \] (2)

Bearing strength with applied axial compression: \[ V_{\text{beg,N}} = k_0 f'_c A_{\text{beg}} \left( 1 + \frac{N}{f'_c A_{pl}} \right) \] (3)

The bearing area will be contributed to by the shear lug, by the embedded plate (if present), and by welded anchors (if present). This general linear model is further developed below.

### 3.1 Basic bearing strength in shear

The basic bearing strength is determined from the tests performed by Rotz and Reifschneider\textsuperscript{3,4} using the BU, CU, and DU test series which had surface mounted plates so only the shear lug and anchors were in bearing at the time of bearing failure. The contribution of the anchors was subtracted from the failure loads in order to assess the actual bearing stress on the shear lug. This was accomplished by using Figure 6 where the AU test series was for a surface mounted plate with only the anchors. By noting the bearing force in the anchors in the AU2 test at the displacement where bearing failure occurred in tests BU1, CU1, and DU1 the bearing force in the anchors at this displacement could be deducted from the total bearing failure load leaving only the bearing force carried by the shear lug. Using these values of the shear carried by the shear lug, along with the values for bearing area of the shear lugs and concrete strength for BU1, CU1, and DU1 from Table 1, the mean value of \( k_0 \) is found to be 2.2 with a coefficient of variation of 0.06 which results in Eq. (4) for the basic mean bearing strength in shear.

![Figure 6: Shear carried by the shear lug for test BU1, CU1, and DU1](image-url)
Michler Model A verifies this $k_0$ value for the basic bearing strength. Figure 7 shows the type of test setup used for the Michler study. Eq. (5) presents Michler Model A in terms of the bearing capacity in shear and Eq. (6) shows the reduction of the model using a shear load eccentricity, $e_v$, of 50 mm and a concrete strength, $f_{c'}$, of 40Mpa. It should be noted that $F_{a,0}$ is the concrete breakout strength of the anchor in tension and the anchor was padded to transmit no shear forces.

Figure 7: Type of test setup used for the Michler study

\[
V_{brg,0} = 2.2 f'_{c} A_{brg}
\] (4)

\[
V_{brg} = f'_{c} A_{brg} \left[ 2.475 - 0.570 \frac{F}{F_{a,0}} - 0.005 e_v - 0.002 f'_{c} \right]
\] (5)

\[
V_{brg} = 2.2 f'_{c} A_{brg} \left( 1 - 0.25 \frac{F}{F_{a,0}} \right)
\] (6)

As indicated by Eq. (6) for the case of no axial load, Michler Model A gives the same results of 2.2 for $k_0$ as that determined from the Rotz and Reifschneider tests and shown in Eq. (4).

Further verification of the basic bearing strength in shear is provided by taking the 5% fractile with 90% confidence of Eq. (4) assuming a coefficient of variation of 0.15 which yields Eq. (7). Eq. (7) is the same as that provided in ACI 318-14 Section 22.8 for the nominal bearing strength under a base plate in compression with concrete extending away from the base plate.

\[
V_{brg,0} = 1.7 f'_{c} A_{brg}
\] (7)

3.2 Modification factor for applied axial tension

For anchorages with applied axial tension, the anchor tensile strain results in a displacement in the direction of the tension load. Since the shear lug, embedded plate and welded anchors are transferring shear by bearing on the concrete, this displacement causes friction forces to develop at each of the steel/concrete interfaces that are transferring shear by bearing resulting in reduced bearing strength with increased tension (Figure 8).
Eq. (8) provides the linear modification factor for applied axial tension that is applied to the basic bearing strength of each bearing surface as shown in Eq. (9). As shown in Figure 9, which uses Eq. (4) for the basic bearing strength on each bearing surface and the failure loads, bearing areas, and concrete strengths provided in Table 1, this modification factor provides a very good fit to the Rotz and Reifschneider\(^3\) tests with applied tension.

\[
\psi_{brg, +N_u} = 1 - \frac{N_u}{\sum A_{sef, ut} f_{ut}} \leq 1.0 \quad (8)
\]

\[
V_{brg} = \left[ V_{brg, sl} + V_{brg, pl} + V_{brg, a} \right] \psi_{brg, N} \quad (9)
\]

Figure 9: Eq. (8) modification factor for tension compared to Rotz and Reifschneider\(^3\) tests

### 3.3 Modification factor for applied axial compression

For attachments with applied axial compression, the confining force of the embedded plate acts to increase the bearing resistance of the shear lug and welded anchors but does not increase the bearing strength of the leading edge of the embedded plate since the area in front of the plate is not subjected to this confining pressure (Figure 10). The limit on \(\psi_{brg, +N_u}\) is based on the extent of test data and a recommended upper limit of 4.75 for \(k_o\) in Michler\(^5\) Model D.
Eq. (10) provides the modification factor for applied axial compression while Eq. (11) indicates how it is applied to the basic bearing strength of each bearing surface. In addition to the bearing shear strength there is also a shear strength from friction present. Based on Cook and Klingner\textsuperscript{7}, the value of the coefficient of friction can be conservatively taken as 0.4. As shown by Figure 11, which uses Eq. (4) for the basic bearing strength and the failure loads (reduced for friction), bearing area and concrete strengths provided in Table 1, this modification factor provides a very good fit to the Rotz and Reifschneider\textsuperscript{3} tests with applied compression (test series BUC).

\[ \psi_{beg,Nc} = 1 + 5 \frac{N_c}{f_c A_{pl,N}} \leq 2.0 \]  

(10)

\[ V_{beg} = [V_{beg,ul} + V_{beg,a}] \psi_{beg,Nc} + V_{beg,pl} \]  

(11)

3.4 Effective bearing area of shear lugs

The effective bearing area of shear lugs has been studied extensively by Michler\textsuperscript{6}. Unless additional analytical work is performed, his recommendation for the effective area of the shear lug is taken as its width multiplied by an effective depth defined by Eq. (12).

\[ h_{ef} = 0.9 \sqrt{\frac{E I_{beg}}{12 f_c}} \leq h_{beg} \]  

(12)
When the range of shear lug thickness and concrete strength parameters in Eq. (12) is investigated, the effective depth of the shear lug can be further simplified to being taken as twice its thickness as shown in Fig. 12. Figure 13 provides details of how this can be implemented.

![Figure 12: Simplified effective depth of shear lug](image)

![Figure 13: Application of effective depth and area of shear lug](image)

3.5 **Anchor tension due to eccentricity of the bearing reaction on the shear lug**

The possibility of anchor tension resulting from the moment caused by the eccentricity of the bearing reaction on the shear lug needs to be considered when designing for anchor steel strength and concrete breakout strength in tension. Figure 14 shows three conditions where moment from eccentricity of bearing reactions in shear occurs. In each of these conditions, only those forces contributing to moment about the right hand anchor at the surface of the concrete are shown for clarity ($\Sigma M_0$). As shown in Figure 14(a), when summing moments about the right anchor at the surface of the concrete,
all that is needed to prevent tension in the anchor beyond that present from friction is that the spacing of the anchors not be less than $e/\mu \left[ \mu V_s = V_e \right]$. With $\mu=0.4$ and the distance to the bearing reaction on the anchor equal to one diameter this would occur whenever the spacing is greater than 2.5$d$ which is always the case.

Next consider a surface mounted plate with a shear lug but with no additional tension in the anchors as shown in Figure 14(b). For the surface mounted plate with a shear lug, the $e$ distance is half of the effective depth of the shear lug which will be one thickness, $t$, of the shear lug if the effective depth is taken as twice the thickness of the lug. The friction force providing the resisting moment is located at half the anchor spacing less half the shear lug thickness from the right hand anchor. Note that the bearing and friction forces on the anchors are not shown since they have been dealt with in the discussion of Figure 14(a) above. When summing moments about the right anchor at the surface of the concrete, all that is needed to prevent any additional tension in the anchor is that the spacing of the anchors not be less than six times the thickness of the shear lug $[\mu V(s/2-t/2) = V_e]$. This condition was met for Bechtel tests series BU, CU, and DU so for these test series there was no additional tension transferred over the length of the anchor as a result of bearing on the shear lug. Figure 14(c) shows the condition when tension is transferred over the length of the anchor due to the eccentricity of the bearing reaction on the shear lug. For this case the anchor spacing must be less than $6t$.

## 4 Concrete Breakout Strength in Shear

The proposed method for evaluating concrete breakout strength where shear is perpendicular to an edge is very similar to that used for anchors in *fib* Bulletin 58 and in 17.5.2 of ACI 318-14. The only difference is in the determination of $A_{Vc}$ and a simplification for $V_b$. Eq. (13) for concrete breakout of shear lug connections is based on ACI 318-14 Eq.(17.5.2.1b). The modification factors for eccentric load, other edges, and member thickness are the same as given in 17.5.2 of ACI 318-14. $A_{Vc}$ is as shown in Figure 15, $A_{Vco}$ is the same as for anchors and is taken as 4.5$c_{a1}^2$. $V_b$ is based on ACI 318-14 Eq. (17.5.2.3b) that applies to concrete edge failure in shear for anchors $\frac{3}{4}''$ and larger where the flexural stiffness of the anchor does not need to be considered. The value for $V_b$ given in Eq. (14) is mean strength in uncracked concrete.
With shear transferred by the shear lug, embedded plate (if present), and welded anchors (if present), the bearing surfaces all displace the same with any increment in applied shear load. This is similar to connections with anchors welded to the steel attachments where concrete edge failure originates from the row of anchors furthest from the edge. In anchorages with shear lugs, the effective bearing area of the shear lug and embedded plate (if present) dominate over the effective bearing area of anchors further from the edge than the shear lug. As a result, the concrete breakout strength for the anchorage should be determined based on the concrete breakout surface originating at the shear lug (Fig. 15).

As noted in the background section, there have been two large scale tests with concrete breakout failure for shear lugs reported by Gomez et al. In fact, there were actually four tests since the loading was cycled from positive to negative with breakout failures in each direction. Figure 16 taken from Gomez et al. shows the shear lug (bearing width 152mm and bearing depth 76mm and 140mm.), the block dimensions (1" = 25.4 mm), and the breakout failure in both directions. When the proposed method for concrete breakout is applied to these test results, the ratio of the actual strength to the predicted strength for the four tests is 1.40, 1.16, 1.06, and 1.02 with a mean of 1.16 and coefficient of variation of 0.14 indicating a slightly conservative model with a reasonable consistency.
5 Summary

This paper presents a simplified design model to predict both the bearing strength in shear and the concrete breakout strength in shear for anchorages with shear lugs. For bearing strength in shear, the model is based on a basic bearing strength modified linearly for the effects of applied tension or applied compression. The nominal value for basic bearing strength based on a 5% fractile is provided by Eq. (7). For applied tension, the modification factor for tension is given by Eq. (8) and the bearing strength by Eq. (9). For applied compression, the modification factor for tension is given by Eq. (10) and the bearing strength by Eq. (11). The concrete breakout strength is provided by Eq. (13) and Eq. (14). Since Eq. (14) represents a mean, the constants should be multiplied by 0.75 to provide a 5% fractile giving a constant of 5.3 for metric and 12.8 for US units. These values may need to be further reduced if the designer assumes that a crack is present. If anchor spacing is less than six times the thickness of the shear lug then tension in the anchor resulting from the eccentricity of the bearing reaction on the shear lug needs to be considered. For a more detailed analysis of anchorages with shear lugs, the reader is referred to Michler’s Model B that is a comprehensive iterative model with consideration for compatibility of deformations.

6 Acknowledgement

Michler was made possible by the research project CU 37/3-1 "Capacity of mountings with shear lugs in the concrete construction to transfer large shear forces", financed by the German Research Foundation (DFG), and on the preparatory work of Dr.-Ing. Christoph Körner.

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FIXING OF WINDOWS WITH FALL PROTECTION

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ABSTRACT

Glass areas in modern building are becoming ever larger and in refurbishment, window openings are sometimes enlarged. Window elements with fall protection are often used, which undertake the additional function of fall protection, for example of a handrail. Furthermore, opening casements over the entire window height are often installed and "French balconies" of metal are replaced, for example, by appropriate glass panel fillings screwed directly to the window frames. This means that the fixing of the window frame is as important as that of glazing with fall protection. The article describes the current status of this subject and explains fixing into the structure.

1 Introduction

Building has fundamentally changed in many areas in recent years. One very visible change is the ever larger glass areas in modern buildings, or the enlargement of window openings when existing buildings are refurbished (Fig. 1). Window elements with fall protection are often used, which also provide the function of an edge protection such as a handrail.

Furthermore, French balconies or balconettes of metal are often omitted when opening casements are installed over the entire window height. These traditional hand-made external handrails are increasingly being replaced by glass panes screwed directly to the window frame with special fixing constructions (Fig. 2). This means that the fixing of the window frame has the same significance as the already mentioned fall-protection glazing, which "only" consists of frame and pane without any additional handrail. The following discussions therefore also apply to the fixing into the building structure of the windows shown in Fig. 2 with additional "window handrail". This means that the fixing of the window frame is as important as that of glazing with fall protection. The article describes the current status of this subject and explains fixing into the structure.

2 TRAV has now become DIN 18008, Part 4 – Additional requirements for barrier glazing

2.1 General

The complete introduction of DIN 18008-4:2013-07² in 2015 replaced "Technische Regeln für die Verwendung von absturzsicheren Verglasungen" (TRAV) (technical rules for the use of fall-protection glazing) (version of January 2003¹³). DIN 18008-4 shows that the provisions of the TRAV have been almost completely incorporated into the standard.
In general, an edge protection (as fall protection) is necessary according to the German sample building regulations *Musterbauordnung* (MBO, §38) if there is a determined height difference between two circulation areas. Circulation areas are defined as areas, in which persons can circulate (both in public and private areas).

### 2.2 Verification of the load-bearing capacity of fall-protection glazing

DIN 18008-4 requires, just as formerly the TRAV, that fall-protection glazing can always fulfil two verifications of load-bearing capacity:
- Verification of the load-bearing capacity for static actions such as wind, climate, horizontal imposed load (or briefly "rail load", Fig. 3)
- Verification of the load-bearing capacity for impact actions from colliding persons.

Glazing of the Categories C1, C2 und C3 (acc. To Din 18008-4\(^2\), which is used indoors, only requires the verification for impact actions.

**Figure 4: Impact areas for impact-type actions (excerpt) according to DIN 18008-4 (or TRAV)**

### 2.3 Verification of the load-bearing capacity of the immediate glass fixings

The immediate glass fixings, such as clamping strips, glazing rebates, screwed fixings, brackets etc., are also to be verified for load-bearing capacity under impact actions. The verifications under impact loading are performed according to the applicable technical regulations, for example for metal construction according to DIN EN 1993\(^{10}\) or DIN EN 1999\(^{11}\).

Verifications under impact loading are performed according to DIN 18008-4, Annex D, "Verification of the impact resistance of support constructions". These verifications can be performed by calculation under the applicable technical regulations or if this is not possible, experimentally. In addition to the verifications of load-bearing capacity described here under static and impact actions, fall-protection glazing also has to be verified for the serviceability state, i.e. maintenance of the permissible deflection.

### 2.4 "Verification chain" for fall-protection window elements

In the design and construction of fall-protection window elements, all parts of the construction have to be considered in the verifications. The term "verification chain" is used for this.

"Verification chain" denotes the verifications of the load-bearing capacity under static and impact actions of the fall-protection glazing (1\(^{st}\) link of the chain), of the immediate fixing (2\(^{nd}\) link of the chain), the parapet rail (3\(^{rd}\) link), the connection of the parapet rail to the window frame (4\(^{th}\) link), the window frame (5\(^{th}\) link) and the fixing or anchoring of the window frame into the building structure, e.g. masonry (6\(^{th}\) link).
Supplementary provisions for the verification of glazing and the immediate glass fixing (links 1 and 2) can be found in DIN 18008-4. The ETB guideline "Building elements, which protect against falling" includes requirements for the further links of the verification chain.

### 3 ETB guideline – Building elements, which protect against falling

#### 3.1 General

The ETB guideline was published in 1985 and is still today valid without revision. The ETB guideline applies to non-load-bearing building elements, which apart from their self-weight are only subjected to loads acting on their surface, which they transfer to other building elements. These building elements also have the task of securing a room or room section that they surround so that persons and objects, which act on these building elements, are not endangered (e.g. safety function against falling).

The guideline is to be applied for building elements, which secure a height difference between circulation areas of more than 1 m. For building elements, which can be assessed as adequately safe from experience, no verification of impact loading has to be performed according to the guideline. The ETB guideline differentiates two installation areas for room-forming building elements, parapets, edge protections and similar:

- Installation area 1: houses, hotels or office rooms etc. with small gatherings of persons,
- Installation area 2: Larger gathering rooms, school rooms etc. with large gatherings of persons.

The guideline also differentiates the loading of building elements, which protect against falling, between on the one hand "horizontal, static loads", and on the other hand "impact loading" (see Sections 3.1 and 3.2 in Ref. 12).

#### 3.2 Horizontal, static loads

For installation area 1, a horizontal load (linear load) of 0.5 kN/m is to be assumed, and for installation area 2 a load of 1.0 kN/m at a height of 90 cm above the floor. For handrails, the load is to be applied at rail height, even if the rail height deviates from 90 cm.

Wind loads are to be superimposed on these loads. According to MLTB Part 1, Appendix 1.3/1, the following two points are to be observed in the application of the ETB guideline:

- If larger horizontal linear loads are given according to DIN EN 1991-1-1 in combination with DIN EN 1991-1-1/NA, these have to be taken into account.

This point is discussed in more detail in Section 5.2.1 of this article.

- The sentence "wind loads are to be superimposed on these loads" is to be replaced by "Wind loads are to be superimposed on these loads, except for parapets of balconies and gallery accesses, which do not serve as escape routes"
Regarding the superimposition of horizontal load and wind load, reference is also made to the statements of Ref.\textsuperscript{21} and Section 5.2.2 of this article.

### 3.3 Impact loading

For the differentiation of persons and objects, which act on fall-protection building elements, the ETB guideline differentiates between the "soft impact" and the "hard impact":

- The soft impact corresponds to an acting impact body mass of 50 kg and an impact velocity of 2.0 m/s. This impact is intended for the verification at the impact location, which causes the least favourable bending in the building element. Superimposition with other loading cases is not necessary (Ref.\textsuperscript{12}, Section 3.2.2).

- The hard impact corresponds to an acting impact body mass of 1 kg and an impact velocity of 4.47 m/s. This impact is intended to verify the impact of a small compact mass at higher velocity in order to be able to assess the behaviour of the building element with localised destruction (Ref.\textsuperscript{12}, Section 3.2.3).

Building elements in installation areas 1 and 2 must not be completely or locally destroyed or penetrated by soft or hard impacts. After each impact, the following conditions have to be maintained:

- The stability of the building element must be maintained.
- The building element must not be torn out of its fixing.
- Broken pieces, which could seriously injure persons, must not fall out.
- The building element must not be penetrated through its entire thickness by the loads defined in the guideline.

The hard impact does not need to be verified for the fixing to the structure. For practical building cases, verification of the soft impact is sufficient. This verifies that the fixing element has a higher resistance than 2.8 kN for this case. The force, under which failure just does not occur, may be taken as the resistance force. Note: In Ref.\textsuperscript{19}, the stated "resistance force" is defined as the failure load and it is required that the fixing shows a static load-bearing resistance of 2.8 kN per fixing point.

### 4 Fixing to the structure

It should be self-evident that the requirement of the standard for verification of the stability of the glazing of a window should also include the verification of the transfer of the forces to be restrained into the load-bearing anchoring substrate (building element).

For DIN 18008-1\textsuperscript{1}, a similar formulation was chosen; it states as follows in Section 8.1.1:

"For the verification of the glass fixing, sub-construction, fixing to the building etc., the applicable technical regulations apply."

The assembly guideline\textsuperscript{19}, Section 5.3.2 also makes clear with reference to fall-protection elements:

"The load transfer from the fall-protection building element to the load-bearing substrate must be verified." It also defines that "fixing systems [...] with a national technical approval under building
The design of glazing according to DIN 18008-4 also implies that the load transfer of actions on the fall-protection glazing and the "verification chain" described in Section 2.6 in principle has to be followed and verified from the action location to the load-bearing substrate and it is not permissible simply to interrupt the verification at the fixing or permit it to be interrupted. As mentioned in the previous paragraph, only approved fixings or anchor systems or fixings regulated by a single-case approval can be structurally analysed. For building structure connections to other building materials, the verification has to be performed according to the applicable technical regulations, for example according to DIN EN 1993\textsuperscript{10} in steelwork.

Above all for fixings in masonry construction, built for example with blocks with very thin webs and low compressive strengths, it can be very difficult to verify the necessary design loads from the requirements for fall-protection glazing for the combination of anchor and anchoring substrate.

5 Verification of the fixing

5.1 General

In a letter of 02/12/2014\textsuperscript{23}, the Building ministers' conference described how the anchoring to the building (Fig. 5) is to be designed. "The structural stability of building elements, which provide safety against falling, is to be verified using technical building regulations. Section 6.4 of the standard DIN EN 1991-1-1/NA:2010-12 contains statements about the horizontal loads for fall safety. The transfer of horizontal loads to provide safety against falling into the load-bearing elements of the building is to be verified. Only regulated building products and products with a general verification of suitability can be considered. It should be pointed out that the ETB guideline intends supplementary verifications against impact loading in addition to the verification [...] mentioned above."

![Figure 5: Examples of fixing points for the transfer of impact-tape loading according to the ETB guideline](image)

One way of complying with the requirements in Ref.\textsuperscript{23} is the use of fixing solutions that are approved as a system, such as the "Window mounting bracket W-ABZ" (Figures 6 and 7). This system has a national technical approval\textsuperscript{14}, which covers under building regulations both the rail itself and fixing
at the steel profile of the plastic window frame and the use of anchors in the anchorage substrate. In this system, plastic anchors with an approval for multiple use for non-structural applications can also be used for this special application, since the system approval newly regulates the application in this special case.

Figure 6: Window installation bracket with national technical approval\textsuperscript{14}

Figure 7: System with national technical approval of anchor, window mounting bracket and screws to fix the window frame to the anchorage substrate for fall-protection glazing

5.2 Verification of the horizontal, static loading

5.2.1 Loading assumptions for horizontal linear loading

In the ETB guideline\textsuperscript{12} the horizontal linear loads were differentiated for installation areas 1 and 2. The modern standard DIN EN 1991-1-1:2010-12\textsuperscript{6} with the associated national Annex DIN EN 1991-1-1/NA:2010-12\textsuperscript{7} are decisive for the "categorisation of areas" (see in contrast also Section 3.2 of this article). For areas with gatherings of people such as concert halls, a horizontal imposed load of 2.0 kN/m even has to be assumed according to Ref.\textsuperscript{7}.

According to DIN EN 1991-1-1/NA, Table 6.12DE, foot note (2)\textsuperscript{7} , the horizontal imposed loads are to be assumed in the full magnitude in the fall direction and in the opposite direction with 50 %, subject to a minimum of $q_k = 0.5$ kN/m.

Thus the corresponding fixings in the area of the rail are to be designed for the following action ($F_{ED}$) from the horizontal linear load: $F_{ED} = q_k \cdot \gamma_Q$ [kN]

with: $q_k =$ according to DIN EN 1991-1-1/NA:2010-12, Table 6.12DE\textsuperscript{7}

$\gamma_Q = 1.5$ independently changeable actions with unfavourable effects according to DIN EN 1990/NA, Table NA.A.1.2(B)\textsuperscript{5}

If the building is a workplace, the Technical rules for workplaces ASR A2.1\textsuperscript{20} (see also Section 2.3) also have to be complied with in addition to the provisions under building regulations. In this case a horizontal imposed load of 1.0 kN/m is always to be assumed for the general case of edge protection.
For Category B1 according to Ref. 7, e.g. office corridors (= workplace), with a horizontal linear load of 0.5 kN/m and Ref. 20 with 1.0 kN/m are thus contradictory. In this case the calculation should assume $q_k = 1.0$ kN/m to remain on the safe side until the two regulations have been coordinated.

### 5.2.2 Superimposition of horizontal linear load and wind load

Fall-protection glazing will in most cases be in external windows, which are loaded by wind loads in addition to the horizontal linear loads. The wind loads and the horizontal linear loads are to be superimposed according to Section 3.2 (exceptions – see also Section 3.2).

The W-ABZ system presented in Section 5.1 can according to Ref. 14 also be designed for wind loads, i.e. the corresponding fall-protection element can be fixed all round with the system, in which case the fixings have to be appropriately verified (see also design example in Section 6 of this article).

The information in Ref. 21 describes how the load superimposition is performed according to DIN EN 1990/NA, NCI to 6.4.3.2(3). According to this, the combination factor for the horizontal linear load (rail load) $\Psi_0 = 0.7$ and for wind loads $\Psi_0 = 0.6$ (see Ref. 5, Table NA.A.1.1). Thus at least two loading cases have to be investigated:

1. The rail load is assumed fully and the wind load is reduced by the factor $\Psi_0 = 0.6$.
2. The wind load is assumed fully and the rail load is reduced by the factor $\Psi_0 = 0.7$.

For the verification of the wind load and the horizontal linear load, however, two separate approaches are conceivable, in which in the opinion of the authors the full wind load and half the rail load should be considered to be on the safe side:

- on the one hand, the window fixing can, for example, be completely constructed according to Ref. 19, which means that the window is fixed and verified according to the provisions of Ref. 19 with regards to wind loads, see Ref. 8 and Ref. 9.
- In addition, fixing systems like the W-ABZ system described in Section 5.1 can be used to resist the rail loads. The verification of the rail load is then performed for the approved fixing system according to the corresponding approval 14 or the stated European codes.

Tables 1 and 2 show as examples the possible window widths when the actions from Ref. 7 and the load-bearing capacity of the wall anchor system W-UR 8 according to Ref. 16 are considered. For higher loads it is clear that two brackets can be needed at rail height to resist the horizontal imposed load (Fig. 8). These brackets should then be arranged symmetrically at rail height or the assumed height of the horizontal imposed load. It should be noted in this case that the steel profile in a PVC window has to be suitable for the load transfer to two brackets and that the appropriate spacing of the anchor system, that is the spacing between two brackets according to the appropriate approval, is maintained.

![Figure 8: Examples of fixing points for the transfer of horizontal imposed load with two brackets at rail height](image)
Table 1. Maximum window widths for the resistance of the horizontal imposed load \( q_k = 0.5 \, \text{kN/m} \) according to DIN EN 1991-1-1/NA\(^2\) e.g. for private living rooms and occupied rooms

| Anchor system W-UR 8mm | (ETA-08/0190) | Art. No.: 0912 808 802 (\( h_{\text{nom}} = 50 \, \text{mm} \))  
| Art. No.: 0912 808 803 (\( h_{\text{nom}} = 70 \, \text{mm} \)) |  |
|---|---|---|
| **Anchorage substrate** | Block dimensions | Edge spacing  
| &nbsp; |  | \( c_{\text{min}} \)  
| &nbsp; |  | \( h_{\text{nom}} \)  
| **Maximum window width** | ETB Guideline fulfilled? |
| Concrete \( \geq \text{C12/15} \) | 70 | \( > 50 \) |  |
| Concrete \( \geq \text{C16/20} \) | 50 | \( > 50 \) |  |
| Silka XL Basic KS \( \geq 28 \, \text{N/mm}^2 \)\(^1\)) | \( \geq 248 \times 175 \times 498 \) | 50 | 70 | 2.5  yes \(^3\) |
| Aerated concrete AAC \( \geq 7 \, \text{N/mm}^2 \)\(^1\)) | \( \geq 499 \times 175 \times 249 \) | 60 | 70 |  |
| Joint width \[\text{[mm]}\] |  |  | \( \leq 30 \) |  |
| Number of brackets at rail height \[\text{[Pieces]}\] |  |  | 2 |  |

| ASSY 3.0 KOMBI 8x80 mm | (ETA-11/0190) | Art. Nr.: 018420880 |  |
|---|---|---|
| **Anchorage substrate** | Timber dimensions | Edge spacing  
| &nbsp; |  | \( c_{\text{min}} \)  
| &nbsp; |  | Minimum screwed-in depth  
| **Maximum window width** | ETB guideline fulfilled? |
| Softwood strength class C24\(^2\)) | \( \geq 120 \times 120 \) | 60 | 73 | 2.5  yes |
| Joint width \[\text{[mm]}\] |  |  | \( \leq 30 \) |  |
| Number of brackets at rail height \[\text{[Pieces]}\] |  |  | 2 |  |

\(^1\) Edge spacing to bed joint or vertical joint according to ETA-08/0190 are to be maintained.

\(^2\) Screw must according to ETA-11/0190 be predrilled 5.0 mm.

\(^3\) \( \gamma_{\text{Mm}} = 1.0 \) (exceptional load case)
Table 2. Maximum window widths for the resistance of the horizontal imposed load $q_k = 1.0 \text{ kN/m}$ according to DIN EN 1991-1-1/NA\textsuperscript{7} e.g. for corridors in public buildings or workplaces according to Ref.\textsuperscript{20}

<table>
<thead>
<tr>
<th>Anchor system W-UR 8mm</th>
<th>(ETA-08/0190)</th>
<th>Art. No.: 0912 808 802 ($h_{\text{nom}} = 50$ mm)</th>
<th>Art. No.: 0912 808 803 ($h_{\text{nom}} = 70$ mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Anchorage substrate</strong></td>
<td>Block dimensions</td>
<td>Edge spacing $c_{\text{min}}$</td>
<td>Anchor setting depth $h_{\text{nom}}$</td>
</tr>
<tr>
<td>Concrete $\geq$ C12/15</td>
<td></td>
<td>70</td>
<td>$&gt; 50$</td>
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<tr>
<td>Concrete $\geq$ C16/20</td>
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</tr>
<tr>
<td><strong>Joint width</strong></td>
<td></td>
<td></td>
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<tr>
<td><strong>Number of brackets at rail height</strong></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>ASSY 3.0 KOMBI 8x80 mm</th>
<th>(ETA-11/0190)</th>
<th>Art. No.: 018420880</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Anchorage substrate</strong></td>
<td>Timber dimensions</td>
<td>Edge spacing $c_{\text{min}}$</td>
</tr>
<tr>
<td>Softwood strength class C24 $^2$</td>
<td>$\geq 120 \times 120$</td>
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</tbody>
</table>

$^1$ Edge spacing to bed joint or vertical joint according to ETA-08/0190 are to be maintained.

$^2$ Screw must according to ETA-11/0190 be predrilled 5.0 mm.

$^3$ $\gamma_{Mm} = 1.0$ (exceptional load case)
5.2.3 Use of plastic anchors

The use of the W-ABZ system includes a requirement to use the plastic anchor W-UR 8 according to Ref. Without this regulation according to Ref., plastic anchors with a European technical approval/assessment, which are approved for multiple use for non-structural applications, cannot simply be used in this application for the fixing of fall-protection window elements. Further regulations for the use of this type of anchor system are contained, for example, in Ref.

In contrast to the provisions of the plastic anchor approval, which only apply for multiple use for non-structural applications under loading from all directions (tension, shear load and combined tension and shear loading), the provisions of Ref. only apply for pure shear loads in combination with the use as fixings of fall-protection window elements, which is a reason that a deviation from the provisions of the plastic anchor approval is possible here. The approval in this case represents a new suitability verification of the plastic anchor under building regulations in this application.

With plastic anchors, a spacing of at least 25 cm (see Ref.) also has to be maintained between the brackets in the masonry anchorage substrate in order that each anchor (and thus each bracket) can be exploited with the full load-bearing capacity of the anchor. If the spacing is less, the anchors may not be considered as "single anchors", meaning that a bracket at this height would be useless. Only in concrete are smaller spacings possible for the use of single anchors.

For the case that the anchor load-bearing capacity is still insufficient for the corresponding masonry, it can be sensible in masonry to consider the use of injection systems with a European technical approval according to Ref. for anchoring.

5.3 Verification of impact loading

The ETB guideline was produced at a time long before the current code series of the Eurocodes, which do not consider any action from impact on a fall-protection window element. At the time of publication of the ETB guideline, the current safety factor concept of partial safety factors had not yet been introduced, which makes its implementation today with integration of the guideline into a design process under current codes difficult, particularly as the ETB guideline is expressly inapplicable for building elements of glass (see MLTB, Part 1, Appendix 1.3/1).

In der ETB guideline, as has been described in Section 3.3, the verification of the fixing elements is to be performed for a "resistance force" of 2.8 kN as an equivalent static load from a soft impact. At the time of publication of the guideline, a global safety concept was in use, although the guideline...
itself does not state any safety factors. Given this, it would be possible to set an exceptional action according to DIN EN 1990/NA with a partial safety factor of $\gamma_F = 1.0$ in the course of a design performed today, since only complete failure or falling out has to be avoided (see Section 3.3).

Considering the terms that are current today, the 2.8 kN stated in the ETB guideline can be interpreted as a characteristic action and given a partial safety factor of $\gamma_F = 1.0$ also as a design value of the action.

At the time of publication of the ETB guideline, there was also no European approval system for anchors. Anchors were also evaluated according to a global safety factor system at that time. The guideline also contains no instructions for dealing with such products. In most approvals for anchors, there is a note that anchors can only be designed for static or quasi-static actions. The guideline ETAG 020 for the assessment of plastic anchors in the course of a European approval procedure excludes impact actions if they occur as dynamic actions. The action from the ETB guideline is however no dynamic action since a one-off impact is assumed. Therefore describes the action from the ETB guideline as an "equivalent static load". Dynamic actions can according to DIN EN 1990:2010-12, Section 1.5.3.13 be considered as quasi-static actions if the dynamic action (here as impact) can be described as an equivalent static load. This makes it possible to design the anchor appropriately in this case.

Verification of the action of an impact load (soft impact):

$$\frac{F_{Ed}}{F_{Rd}} \leq 1.0$$

with:

$$F_{Ed} = F_{Ek} \cdot \gamma_A$$

$$F_{Rd} = F_{Rk}/\gamma_m$$

$F_{Ek} = 2.8$ kN resistance force according to the ETB guideline, Section 3.2.2.2.3, [12] or see Section 3.3

$\gamma_A = 1.0$ partial safety factor for exceptional actions according to DIN EN 1990/NA, Table NA.A.1.2(B)

$\gamma_m = 1.0$ partial safety factor for the anchorage substrate

6 Masonry with low load-bearing capacity

Tables 1 and 2 show that fixings in calcium silicate and aerated concrete blocks with higher compressive strengths can be verified without problems with a plastic anchor W-UR 8. This applies both for the horizontal imposed load up to $q_K = 1.0$ kN/m and also for the anchoring of the equivalent static load from a soft impact according to the ETB guideline. With masonry of low compressive strength, this verification for an anchor is generally no longer possible, since the action for the soft impact from the ETB guideline can no longer be resisted by one anchor only.
Due to the sometimes low load-bearing capacity of anchors in various walling materials, is can however be necessary for the design of the anchors to distribute the equivalent static load onto several fixings into the anchorage substrate. In the ETB guideline, this is referred to as a "fixing element". In current terms, this would either be understood as a single anchor or as a fixing location with several anchors (e.g. a group of two or four anchors). A possible distribution of the loading onto one "fixing element" with two anchors is shown in Fig. 9.

7 Summary

The importance of fall-protection window elements is increasing ever more in modern new building and refurbishment. At the same time, there are no definite current regulations for dealing with fixing into the structure and its structural design, since the current guideline is more than 30 years old and no longer fits the current European design codes (Eurocodes).

This article is therefore intended to describe the current situation and offer some solution approaches in the view of the authors, how fall-protection window elements can be fixed into the structure and what constraints have to be taken into account for their structural design.

References:


13. Deutsches Institut für Bautechnik (DIBt), Technische Regeln für die Verwendung von absturzsichernden Verglasungen (TRAV), Fassung Januar 2003.


18. EOTA: ETAG 029, Leitlinie für die europäische technische Zulassung für Metall-Injektionsdübel zur Verankerung im Mauerwerk, April 2013, Brüssel.


These days adhesives are used in large quantities in the construction industry and also in many other industrial applications. Annually a global trading of approximately 60 billion euros with adhesive bonding, sealants, adhesive tapes, etc. is processed. Alone in Germany more than 800 thousand tons of adhesive were produced in 2015. The German construction industry is currently processing at an average 50 thousand tons of adhesive per year while the German paper and packaging industry is processing 126 thousand tons and the woodworking industry 32 thousand tons per year. Due to the variety of adhesives and specialty of each adhesive, it is hardly possible today that all the adhesives along with their application requirements are included fully in the data sheets. However, in order to ensure reliable and lasting adhesion, not only the correct selection of the adhesive is important, but the surface treatment of the materials and appropriate application are also equally critical. Adhesive anchors on component surfaces, in particular concrete, are one of the most recent developments in the field of fastening technology. Adhesive anchors are glued on to the concrete, transfer external loads on to the concrete surface and not inside concrete. Compared to the centric tensile force, the tensile adhesion cannot be deduced by the compression strength of the concrete alone. The tensile adhesion is influenced by several parameters such as the form of the surface, releasing agent, compaction, curing or contamination of concrete.

1 Introduction

Structural components of different materials and types invariably require some kind of connection or joining arrangement for formation of structural sub-assemblies, and further the overall structure. The joining of these components to produce the overall structure is referred to as joining or connection technology. The mechanism of force transfer from component to component can be of positive locking, force closure or adhesive bond. Often, two or all the three mechanisms act simultaneously in certain cases.

Figure 1: Classification of the joining technologies - positive locking, force closure, adhesive bond
Generally speaking, a distinction must be made between detachable and non-detachable connections. Perforated connections with screws are extensively applied in steel and timber construction, since assembly at the construction site is fast, easy and the connection is fixed instantly and can also be removed with ease. These features are especially important for temporary structures. Detachable connections are predominantly used for dismantling, maintenance, repair and recycling. Welding and adhesive bonding belong to the permanent substance-to-substance bonds\(^2\). In Case of bonding, two materials adhere to each other by an additive glue. The tightening forces between the atoms or the molecules of the joined parts and the cured adhesive leads to the cohesion of the compound. Generally, adhesive bonds are not designed by assuming a concentration at a point as in case of screws or linear distribution as in case of welding. The adhesive is usually applied to the entire surface available. For this purpose, different adhesives are available, depending on the material characteristics of the surfaces required to be bonded. In construction industry, the most frequently used adhesives are wood glue, silicones and epoxy resins\(^3\).

2 Applications in construction engineering

State of the art, adhesive connections used in wood constructions and glass construction as well as in fastening technology have technical approvals for construction applications. Structural glazing facades, composite dowels and bonded reinforcing bars are some of the examples of application of these connections in the construction industry.

![Figure 2: bonded reinforcing bars](image)

Presently in masonry construction, the laying of bricks (stones) is no longer done with mortar. Instead the bricks (stones) units are glued (bonded) with thin-bed mortar or foamed adhesive. The stones are made of sand-lime sandstone or aerated concrete and vertically perforated sand bricks. These stones allow very thin joints due to their production tolerances and thus can be effectively laid by gluing (bonding). In structural steel, constructive gluing (bonding) is less common. For steel connections, rivet, nail, bolt, screw and weld joints are more popular. In concrete and reinforced concrete, gluing (bonding) is mainly used in strengthening and retrofitting applications to buildings and also for building maintenance. In retrofitting and strengthening applications concrete structures are typically reinforced with CFRP sheets (carbon fiber reinforced polymer), which are required to be glued to the concrete surface with epoxy resin adhesives for an effective load transfer mechanism. CFRP reinforcements can be applied in locally weak zones, for example, in the case of intermittent ceiling perforations, in the case of ceilings and beams for bending and shear reinforcement, as well as for columns by a confinement. CFRP sheets are tapes that can be loaded (stressed) only in one
direction. In the case of a bi-axial load, such as occurs with solid plates, the members are required to be glued (bonded) crosswise. CFRP sheets usually have a thickness of 1 to 2 mm and can be glued (bonded) together in several layers⁴.

3 Strength and durability of an adhesive bond

Adhesive bonding is a material-bonding process with which any materials can be bonded together. The strength and durability resulting from adhesive bonding is influenced by several factors. An adhesive bond is a composite system whose properties depend on the characteristics of the adhesive, the joining surface, the geometric design of the adhesive bond and the acting stress (load).

![Figure 3: Influencing parameters affecting adhesive bond](image)

The quality of adhesive bonding depends on the adhesion of the adhesive to the bonding surface, the adhesion strength of the material, the design of the adhesive bond and the type and direction of the stress. A strong and permanent adhesive bond is possible only with due consideration for all these influences.

3.1 Adhesive

The adherence of surfaces of the same or different materials is referred to as adhesion. Depending on the type of adhesive and surface, the strength of physical forces and chemical bonds may vary. Clean and non-greasy surfaces generally improve the adhesion of the adhesive. The strength of the adhesive itself is referred to as cohesion. The cohesive force is adhesive-specific. Most adhesives are polymers and therefore the influence of temperature on the strength and deformation behavior is particularly strong. Adhesive bonds are only stable within a certain temperature range. At higher temperatures, the adhesive gradually loses its strength and fails. Therefore special protection measures must be considered in particular for possible fire exposures. Adhesives undergo ageing because of moisture, especially in combination with elevated temperatures. If water penetrates into the adhesive layer and/or to the interfaces of the glued materials, the adhesion is weakened or in certain cases the adhesive is dissolved. The water can, on the one hand, act as a plasticizer on the adhesive and, on the other hand, split up chemical bonds. Occasional water contact is uncritical. Therefore, adhesive bonds should be carried out in such a way that they are not exposed to high and permanent moisture⁵.
3.2 Material and surface

The material surfaces are not ideally flat and smooth, but always have a roughness. The actual surface area is always greater than the calculated area when accounting the surface irregularities in a micro level. The adhesion forces in this consideration act in parallel as well as perpendicular to the component surface. Very rough surfaces can form a mechanical bond with the adhesive, but this does not necessarily lead to better adhesion³.

![Figure 4: Surface roughness = geometric deviation from the plane surface](Image)

In general, a rough bonding surface is better in the case of hard, stiff adhesives such as epoxy resin. For flexible, soft adhesives such as silicone, a better adhesion is achieved with a smooth bonding surface⁶.

In event of small thickness, surfaces of materials can deviate significantly from the structure and the chemical composition of the actual material. Pure material characteristics are therefore not sufficient for assessing the adhesion bond strength of a material. The adhesion between component and adhesive takes place in the surface and the external load is introduced into the component through the surface. The surface or the outer layers may have reduced strengths or may bond poorly to the core material. Often, surfaces have non-visible impurities that adversely affect the adhesion strength. In order to improve the strength and quality of adhesion, surfaces are cleaned with appropriate methods³.

3.3 Geometric of the gluing

In view of the constructional aspects, not only the right adhesive on an ideal surface guarantees a firm and lasting bond, but the arrangement of the joining parts and the formation of the adhesive must also be technically correct. Adhesive bonds must always be designed in such a way that the stress does not lead to peeling or splitting of the adhesive layer. Peeling or splitting loads at the end of an adhesive connection leads to low load carrying capacities, because the adhesive layer is loaded uni-directionally instead of load in two dimensions.
Adhesive bonds with metallic joining parts should not be carried out as butt joints, since they have little adhesive surface and are therefore unfavorable for the transmission of force. These bonds should be designed as overlapping bonds, in which the adhesive layer is loaded on shear. This allows larger areas to be activated, which compensates for the lower strength of the adhesive. In general, adequate adhesive surface should always be used for the design and construction of the adhesive bond.

3.4 Mechanical stress on adhesive joints

Under a mechanical load, adhesives have the properties of plastics regardless of the thickness of the joint. Adhesives relax and creep. The relaxation leads to the time-dependent stress reduction in a constantly stressed adhesive joint and creep leads to the time-dependent increase in deformation at a constant load. The creep elongation is composed of an elastic (recoverable) deformation component and a plastic (permanent) deformation component. As with all connections, different load bearing capacities are result as a function of the playing mechanism of load transfer, depending on the load direction (pull, transverse pull) and load type (static, dynamic). The load bearing capacity of adhesive bonds is determined by tests and the evaluated resistances apply only to the tested connections. The test results cannot be generalized for other adhesives. The maximum tensile strength of an adhesive is determined by the adhesion of the adhesive to the joint surface and the maximum shear strength of the adhesive is limited by the strength of the adhesive. Adhesive bonds fail with different failure modes. DIN EN ISO 10365 distinguishes between adhesion and cohesive failure, mixing and surface failure.
4 Adhesive anchors on concrete surfaces

Adhesive anchors on component surfaces, in particular concrete, are one of the most recent developments in the field of fastening technology. Adhesive anchors, which are glued on to the concrete surface, transfer external loads on to the concrete surface and not inside concrete. Compared to the centric tensile force, the tensile adhesion cannot be deduced by the compression strength of the concrete alone. The tensile adhesion is influenced by several parameters such as the formation of the surface, releasing agent, compaction of concrete and by curing or contamination. To realize an adhesive anchor perforated steel plates are bonded with an internal thread or sheet metal with a connecting thread in the form of a welded-on nut with epoxy resin adhesive used in the fastening technique for composite anchors.

Figure 7: Adhesive anchors - steel plate d = 80 mm, bigHead® d = 38 mm

Perforated plates / sheet metal have the advantage that the surplus mortar can emerge laterally and also upwards. This allows the panels to be positioned precisely. They don’t slide laterally during the setting process because of the surplus mortar. In addition, the surplus mortar which enters the perforation, forms adhesive bond with the plate and an interlocking arrangement for shear and tension.

Figure 8: bigHead®, floor-, wall-, ceiling mounting
Steel plates with a diameter of 8 cm and a thickness of 2 cm can be glued to concrete walls or ceilings with thin joints of less than 0.25 mm without slipping or falling. The critical ratio \( \frac{A}{F_{G}} \) of plate area \( A \) (adhesive surface) to plate weight \( F_{G} \) is 7 mm\(^2\)/g for the anchors with epoxy resin to be bonded to a wall or ceiling. It is recommended that this ratio should be higher than 10 mm\(^2\)/g. The load-bearing capacity of C20/25 concrete under centric tension is approx. 2.6 N/mm\(^2\) - assuming that the plates are sufficiently rigid and the plate bending has no influence on the bearing capacity. The failure of these adhesive joints is suddenly caused by the type of failure of the joining part (concrete excavation) and the displacement to the failure of the connection is less than 0.1 mm.

### 5 Conclusions

Today almost all materials in use can be connected to one another by applying adhesives. Since the bonding process is virtually heat-free, the structure of the joining materials is not changed. As a result, even very large joining parts (adherents) can be bonded and virtually any desired material composites can be produced which were till date not so feasible. The reasons for the restricted application in safety-relevant areas of construction are due to the uncertainty in the assessment of the aging, long-term behavior and durability, which are required in the construction industry, particularly with regard to the admission criteria and stability over very long periods. Reliable gluing requires, above all, the possibility of prediction of the expected service life, as well as the knowledge of the adhesive manufacturing processes and influencing factors.

### References:

COMPOSITE ANCHORS TO REDUCE THERMAL BRIDGES IN FACADES

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ABSTRACT

Fixings of rain cladding facades are typically made of metal. This is most usually made with brackets of Aluminum or Steel which have high thermal conductivity. Such thermal bridges have a huge influence on the thermal performance of buildings. By replacing metal brackets with glass fiber composite material, the heat insulation factor can be improved by more than 200 times.

The speech shows a new thermal bridge-free product fastening rain cladding facades onto the building structure. The newly developed Schöck durable anchor system, approved by DIBT, is a glass fiber solution based on the Combar system. All relevant influence factors such as temperature (-40° to +80°C), tensile and shear forces as well as chemical effects have been checked in both short and long term tests. The tests were carried out at the IWB Stuttgart.

With these thermal bridge-free anchors the high energy-saving standards for outside walls can be reached with less insulation material. This solution is not only environmentally sustainable but also cost saving.

1 The system of ventilated rain cladding facades (VHF)

Ventilated facades are high-quality and favorable façade systems. A new market study from the association FVHF in Germany shows the popularity especially concerning the safety, functionality and flexibility of this type of façade. The scope for design is endless in cladding material, geometry and color. Freedom in material and design is a big issue to give every building its own individual character.

2 Thermal bridges within a ventilated Facade

With strengthening of climate protection law and increases in heat-insulation standards, the influence of every thermal bridge will be much more important. Therefore it’s good to have a special view of the material, geometry and number of Anchors as well as the thickness of the thermal insulation. As a result, the thermal quality of the outer wall is defined by the addition of the heat transmission coefficient of the undisturbed wall (U°) and the punctual and linear thermal bridges. (ΔU) [W/m²K]

The lower heat-insulation value, the greater the influence of localized thermal bridging
3 Thermal bridges within a ventilated Facade

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The lower heat-insulation value, the greater the influence of localized thermal bridging

4 New composite materials with less thermal conductivity

Aluminum brackets are very common. The use and installation of those brackets is widely known and used. But the thermal conductivity of aluminum is extremely high; and this has a severe negative effect on the thermal bridges and the wall heat-insulation-value. In situ solutions like thin insulation plates between the brackets and the building wall improve the system somewhat.

Better performance can be reached by the replacement of aluminum with stainless steel. The thermal bridges can be almost completely erased by using facade anchors of glass fiber reinforced plastic
Werner Venter and André Weber

Figure 2: Influence of thermal bridges

(GFRP). GFRP like Combar, for example, have a thermal conductivity less 1 W/mK performances of which is more than 200 times better than aluminum.

\[ \frac{\lambda_{\text{Aluminium}}}{\lambda_{\text{Combar}}} >> 200 \text{ ist.} \]

<table>
<thead>
<tr>
<th>Material</th>
<th>Thermal Conductivity λ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aluminium</td>
<td>200 W/mK</td>
</tr>
<tr>
<td>Stainless Steel</td>
<td>60 W/mK</td>
</tr>
<tr>
<td>Combar</td>
<td>13 W/mK</td>
</tr>
<tr>
<td></td>
<td>0.7 W/mK</td>
</tr>
</tbody>
</table>

Table 1: Thermal conductivity λ

Therefore removing aluminum with Combar reduces thermal bridges, saves energy and reduces insulation thickness. Material and resources are conserved. Thus the wall mountings can be thin-walled and increase the area available for living space.

The following chart shows the required insulation thickness of mineral wool in comparison with 4 types of wall mounting to reach a thermal efficiency (U-value) of 0.24, 0.20 or 0.15 W/m²K. The first column is based on using 3 aluminum brackets, including a thermal stop, per square meter; the second column shows the result calculated using 3 stainless steel brackets and thermal stop; and the third indicates the required insulation thickness when using 3 Combar facade brackets. This allows the same thickness as the undisturbed wall, to be used, as shown in column 4.
New materials have to be utilized

Having looked at the energy calculation results, the big question is: why aren’t composite brackets used as standard?

The answer is clear. The application must allow the use of. All influences like permanent loads, variable actions, thermal actions and chemical effects have to be solutions which last over the lifetime of the facade. In cases where metallic brackets like stainless steel or aluminum are used, there are design standards available. In cases of using composite material, proof of approval has to be demonstrated.

The approval tests including in particular testing by adjust a range of temperatures 80°C down to minus 40°C, including brittleness, as well as the creep characteristics of the plastics. The test results
in all load directions like pressure, tensile and shear, in both short term and long term tests, show the material properties do not degrade over the lifetime of the system.

The building regulations demand the fire protection of material. Under fire protection laws it’s not permitted to use flammable materials in high-rise or public buildings. Currently available thermoplastic material doesn’t achieve that. Using metallic or mineral material is therefore the ideal solution.

6 Schöck Thermoanker the model for a new VHF anchor

Since 2009 Schöck has been selling thermal bridge free anchors for concrete sandwich walls. This special anchor comes out of the glass fiber technology developed in the early 2000 called Schöck Combar. The composite material known as Combar is approved in several countries for use in concrete reinforcement. (Durable 100 Years). One of the most interesting properties is the lower thermal conductivity properties. Therefore Combar is perfectly convenient to be used as tensile rebar in balcony elements thereby eliminating thermal bridges. All Schöck products have been certified and granted approvals. Schöck Thermoanker in particular was the benchmark in developing the new façade fixings.

![Figure 5: Concrete sandwich wall with Schöck Thermoanker](image)

7 Connecting of metallic material to Combar

One important property which makes the use of Combar as a part of a façade sub construction possible is the ability to connect steel profiles and Combar. Together a lot of variations of connection are possible but the best solution to transfer the façade loads into the glass fiber composites is a centric connector. For that a special threaded connector for thermoset material has been developed.
Figure 6: Schöck façade anchor

Chart 1: Short-term +25°C connector Combar 12

Chart 2: Short-term -40°C connector Combar 12
Werner Venter and André Weber

Chart 3: Short-term +80°C connector Combar 12

<table>
<thead>
<tr>
<th>Short-term tensile test</th>
<th>Nu,mean +25°C</th>
<th>Nu,mean -40°C</th>
<th>Nu,mean +80°C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Connector – Combar 12</td>
<td>14.23 kN</td>
<td>13.81 kN</td>
<td>13.80 kN</td>
</tr>
<tr>
<td>Connector – Combar 16</td>
<td>21.80 kN</td>
<td>21.05 kN</td>
<td>20.86 kN</td>
</tr>
<tr>
<td>Connector – Combar 20</td>
<td>25.15 kN</td>
<td>26.45 kN</td>
<td>23.46 kN</td>
</tr>
</tbody>
</table>

Table 2: Summary short-term and long-term results Connector – Combar

The results show only a slight influence of temperature on the tensile resistance between connector screw and Combar.

Chart 4: Long-term Connector-Combar 12
Plastics as well as composites are susceptible to a loss of material properties during the aging process. Therefore it’s important to evaluate the value for degrading. Typically it’s done by long-term-failure tests. After these tests it’s possible to extrapolate to 50 years durability.

The same data and tests are trenchant for evaluating the shear force capacity. Three failure scenarios are possible. First is to determine the shear force effects on steel at the point where it is embedded in Combar. Second, the resistance of composites splitting in cases where shear loads occur and thirdly the combination of bending and pullout failure between Combar and connector.

All of the tests were done with chemical mortar fixed in concrete C20/25 hv=40mm. The set of tests S0 show the results of shear force capacity without lever-arm; S14 show the results with 80mm lever-arm; and S16 with 300mm lever-arm.

Chart 5: S0-Sheartest connector Combar 16 without lever-arm (steel failure)

Chart 6: S14-Sheartest Connector-Combar 16 with 80mm lever-arm (splitting)
Werner Venter and André Weber

Chart 7: S16-Sheartest Connector-Combar 16 with 300mm lever-arm (stop at 48mm deflection)

The test data for S16 with 300mm lever-arm show the typical deflection of linear-elastic material. It also shows the comparative calculation of a good correlation to the curves S16. The calculation is done with E-modules of 60 GPa and length of cantilever $h = \text{lever-arm } l + \frac{1}{2} \text{Combar diameter}$. 

$$h = l + \frac{1}{2} Dn$$

$$F(w) = \frac{3EI(w)}{h^3}$$

$$F_{\text{cal}} [kN]$$

$F_{\text{lost}} [kN]$
8 Buckling in case of pressure

The linear-elastic properties are also the basis for the calculations of the buckling resistance. Characteristic Combar pressure is $f_{ck} = 265 \text{ N/mm}^2$.

\[ F_d = \frac{A \cdot f_{ck}}{\gamma_{m.f}} \]  
\[ N_{el} = \frac{\pi^2 \cdot E \cdot l}{\gamma_{m,E} \cdot E_k} \]  
\[ N_d \leq N_{cr} = \frac{F_d}{1 + \frac{F_d}{N_{el}}} \]

Figure 6: Buckling Combar 12, University of Kaiserslautern

9 Fixing of Combar in concrete

The fixing of Combar is achieved through bonding with chemical mortar. The compatibility between the chemical mortar (vinyl-ester based) and Combar has already been proven tested. All relevant tests according ETAG 001 Part 5 were carried out to allow measurement at the characteristic forces (Option 1). One special feature is noticeable. All tests and values in every diameter are determined by $hv=40\text{mm}$.
The characteristic value of bond strength:

<table>
<thead>
<tr>
<th>Characteristic bond strength, mortar</th>
<th>$\tau_{RK}^{0}$</th>
<th>5.5 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic bond strength, mortar 80°C</td>
<td>$\tau_{RK(80\degree)}$</td>
<td>2.0 N/mm²</td>
</tr>
<tr>
<td>Characteristic bond strength, mortar 25°C</td>
<td>$\tau_{RK(25\degree)}$</td>
<td>2.8 N/mm²</td>
</tr>
</tbody>
</table>

Table 3: Characteristic bond strength, mortar

All relevant calculation is carried out in accordance with EOTA TECHNICAL REPORT TR029. This technical report is widely known and accepted so it’s not part of this paper.

10 Conclusion

The use of new materials and new geometrics in construction opens up opportunities to meet the higher requirements of climatic protection and energy saving. In cases of ventilated façade fixings new glass composite materials are very interesting especially with proven durability and usability with fire protection requirements. The new product Schöck Combar façade fixing meets all necessary requirements.

Figure 7: Wall construction with Combar und GIP adapter

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3. Schöck Thermoanker Zulassung Z_21.8_1894
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THE DYWIDAG DUCTILE CONNECTOR—
AN INGENIOUS CONNECTION METHOD FOR PRECAST
STRUCTURES IN HIGHLY SEISMIC ZONES

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ABSTRACT

The DYWIDAG Ductile Connector (DDC) offers an innovative solution especially for high-rise frame-buildings that are constructed in highly seismic zones. All precast elements can be connected quickly by bolting, which eliminates any time, cost and staff consuming casting on site. The high-strength bolts are inserted through a connector plate and threaded into the key component of the connection, a steel rod embedded inside the column. The rod is made of high-performance ductile steel and accommodates post-yield system deformations. This desired behavior is achieved analytically by dimensioning all other components of the connector using capacity design and was proven in various tests. As a result, the energy induced into the system by the earthquake will be dissipated and the load reduced on all other, less ductile parts of the frame.

Shear forces will be resisted by friction. Frictional resistance is generated by pretensioning of the high strength bolts and by a passively activated compression load that is flexurally induced. Consequently, the need for permanent corbels is eliminated. By using high strength reinforcement within the beams, congestion and resulting installation times can be reduced immensely. Numerous dynamic subassembly and full-size tests have proven the superiority of the DDC to monolithic structures, and the connector has been successfully applied at various projects mainly in California and Asia for many years.

1 Introduction

The precast concrete industry has a reputation for innovative solutions. Their numerous benefits usually result in more economical concepts when compared to cast-in-place or pure steel structures, but in order to take into account all advantages, out-of-the box thinking is often required. However, the application of precast systems in earthquake-prone regions is an enormous ongoing challenge for any designer, as the connections between components are naturally the weakest links in precast concrete structures.

Many severe earthquake events have proven that existing precast buildings could not always resist the high lateral loads, mainly due to unsuitable connection methods. The complete collapse of the
precast concrete parking structure of the California State University during the Northridge earthquake in 1994 is probably one of the most famous examples.\textsuperscript{1}

Already before the disastrous Northridge earthquake, the Uniform Building Code (UBC) contained strict detailing requirements which only permitted the design of equivalent monolithic (emulative) frames with strong connections\textsuperscript{1,2} and which were tightened even further directly afterwards. Due to these normative regulations, mainly traditional monolithic cast-in-place structures or a combination of precast and lateral supporting cast-in-place systems were used in highly seismic regions within the United States in the 1990s.\textsuperscript{3} The numerous advantages of precasting, such as increased construction speed on site due to prefabrication, a higher quality of structural concrete members and the reduction of necessary formwork, which usually results in immense cost and time advantages in comparison to cast-in-place structures, could therefore, if at all, only be partly exploited.

In order to benefit from all mentioned advantages of precasting while achieving a seismic performance which is equal or superior to comparable monolithic structures Englekirk & Nakaki, Inc. started the development of the DYWIDAG Ductile Connector (DDC) in conjunction with DYWIDAG Systems International. Consequently, lateral load resisting frames could be used in precast concrete constructions. The general concept, design, and successful testing of this connector as well as further variants of this connection method are described in detail in the following chapters.

2 Development of the Connector

2.1 Objectives

The primary objective of the DDC development was to create a flexible, functional building that would survive the lateral displacement demanded by the ground motion during an earthquake of catastrophic magnitude. However, even after such catastrophic events, the system must still be fully functional and, if performance based design is used, must comply with the performance level of Immediate Occupancy. In addition, the erection process must be as simple as possible and must not require any special knowledge or unusual quality control measures. For this reason, installation tolerances have to comply with standards accepted by the construction industry. Furthermore, flexibility is essential as the connector may not govern building function or aesthetics in any way; otherwise, its potential applications might be limited. Obviously, in order to be applied at all, the system must be more economical than traditional cast-in-place and steel structures.\textsuperscript{3}

The DYWIDAG Ductile Connector displayed in Figure 1 attained all of the objectives mentioned above, and a reliable, competitive connector was successfully introduced to the building market.

2.2 General concept

At the beam-to-column interface, a connector is introduced that can be made to behave in a ductile manner. As the link will control the loads that are imposed on the rest of the structure, excellent seismic performance can be achieved, and the danger of creating a brittle, weak link at the interface can be avoided in a clever way. This opens up the opportunity to apply the most cost and time effective precasting type in areas of high seismic activity; namely, the independent fabrication of all precast components and simple joining by bolting on site without the need for permanent corbels, wet joints or welding.\textsuperscript{4}
The ductile connector rods are embedded within the precast column at the casting yard, transported to site and erected (Figure 2 left). Beams are also precast with connector plates at the bottom and top part of the beam ends. Each connector plate has a threaded part, which receives DYWIDAG high strength reinforcing threadbars with a tensile strength of 150 ksi [1040 N/mm²] and which becomes the main flexural reinforcement of the beam. Additionally, each connector plate has oversized holes, through which high strength bolts can be inserted on site. In order to allow for the insertion of these bolts, a recess needs to be provided in the beam behind the connector plate (Figure 2 middle). On site, the precast beams can be lifted into position and by screwing the bolts through the connector plate inside the threaded portion of the ductile rod, beam and column are joined together (Figure 2 right). After pretensioning of these bolts full seismic capacity of the frame is attained instantly. The remaining recess of the beam can be filled with grout in a later stage, but this grout has no structural purpose and serves primarily to protect the steel pieces.
The material properties of the yielding ductile connector are well defined with a high elongation capacity that allows strain states above 30%. In order to avoid unintentional overstrength, the yield strength of the ductile rod is strictly controlled with only a very small standard deviation being allowed. By using capacity design with a controlled overstrength factor of 1.25 to dimension the connector plate, the high-strength bolts and the DYWIDAG threadbars, it can be assured that yielding will only take place within the shaft of the ductile rod.

In comparison to conventional ductile frame beams, this relocation of the yielding element from the beam to the interior of the column results in immense structural advantages. At the beam-column interface of traditional cast-in-place connections, high shear and compressive stresses combine at the bottom of the beam. Due to the tensile overstraining of flexural reinforcement at this part of the beam, the now permanently elongated bar buckles when subjected to compressive strains in case of reverse load cycles. Consequently, even small cycles of post-yield rotation at moderate drift angles can result in crushing at the beam column interface. Traditional ductile concrete frames commonly experience such distortion during earthquakes of relatively moderate intensity, because the concrete shell is prone to spalling. This spalling can be discomforting for the occupant in the best and structurally critical in the worst case. By using the DYWIDAG Ductile Connector, this high strain within the beam can be controlled. As yielding of the ductile rod shaft takes place within the column, where the confined concrete provides non-deteriorating lateral support, all imposed shear and flexural loads acting onto the frame beam are limited, and the beam can be designed to behave elastically, preventing bar buckling at low cycles.

By using high strength threadbars as the main longitudinal reinforcement within the beam, congestion can be reduced. Consequently, constructability can be improved immensely, as less construction labor and cost as well as less demand on hoisting equipment at the construction site becomes necessary. Couplers can connect the threadbars, or they can be spliced with standard beam reinforcement.

3 Frame design using DDC

When using the DYWIDAG Ductile Connector as the connection system for precast concrete frames, most of the design and required detailing coincides with all prescriptions contained in the current ACI 318-14 for Special Reinforced Concrete Moment Frames (SRCMF). Hence, a response modification coefficient of \( R = 8 \) may be used for the design. Alternatively performance based design can be applied.

System capacities can be derived from the load transfer mechanism and general conditions of equilibrium. As the ductile rod is the yielding element of the connector, the analytical system evaluation starts at the ductile rod and moves towards the beam and the column.

The total number of provided rods \( N \) and their nominal tensile strength \( T_y \) within the column as well as the distance between the centerlines of the top and bottom longitudinal reinforcement \( (d-d') \) account for the nominal moment strength \( M_n \) of the connection:

\[
M_n = N \cdot T_y \cdot (d-d')
\]
The number of resisting ductile rods as well as the beam depth can be flexibly adapted to fit any design objective. A standard DDC configuration as shown in Figure 1 contains two ductile rods at one connector plate, with a yield strength of 141 kips [627 kN] per rod; hence a total yield strength of 282 kips [1254 kN] per connection. As the design objective for the rest of the frame is elastic behavior, the provided nominal capacity of the ductile rods has to be developed in the beam and a (controlled) overstrength factor of $\lambda_0 = 1.25$ has to be taken into account.

When further following the logical load path, the first transfer point proceeding towards the beam will be the beam-column interface, where the appropriate level of shear and moment has to be transferred by the high-strength bolts that join the precast components together. For the described standard DDC configuration, 1-1/2” [38 mm] A490SC bolts are being used. They provide a nominal tensile strength of 210 kips [934 kN] per bolt, which exceeds the tension strength of the ductile rod by more than 25%. Generally, the required nominal bolt area $N_{AB}$ can be developed from standard LRFD Specifications:

At the beam to column interface, the shear load $V_{nE}$ that has been created by the ductile rods, in combination with the factored dead and live loads according to ACI, needs to be resisted by the connector. The shear load $V_{nE}$ can be derived with the clear span of the beam $L_c$ as follows:

$$V_{nE} = 2\lambda_0 \frac{M_n}{L_c}$$  \hspace{1cm} (2)

All shear loads are being transferred following friction mechanism. The generated load proceeds from the ductile rod face towards the connector plate of the beam through a set of shim plates (see Figure 3). The shear capacity of the connector is a function of the compressive force and the coefficient of friction $\mu$ between the beam and column interface. The larger normal load of either bolt pretension $N \cdot T_p$ or flexurally induced compression $M/(d-d')$ activates this friction load path. Consequently, the nominal capacity of the shear transfer mechanism of the connection $V_n$ is the larger value of the following Equation 3, the detailed derivation of which can be found in Reference 5 and 6:

$$V_n = \text{greater of } (M/(d-d') \text{ or } N \cdot T_p) \cdot \mu$$  \hspace{1cm} (3)

When dead and live loads are the main concern, the friction coefficient $\mu$ should be 0.35. As any shear slip will not result in a system failure, the friction coefficient can be increased to 0.5 when seismic limit state shears are considered.

Further, following the load path, the required area of beam flexural reinforcement $A_{sb}$ can be derived considering the appropriate chosen overstrength and strength reduction factors in order to take any uncertainties of the load path into account:

$$A_{sb} = \lambda_0 \cdot N \cdot T_p / (\phi_b \cdot F_y)$$  \hspace{1cm} (4)

The main longitudinal reinforcement within the beam is provided by the DYWIDAG threadbars, which are threaded into the connector plate. In the standard DDC configuration, two 1-3/8” [36 mm] threadbars are connected to each plate, at the beam top and bottom, but the respective size can again be adapted to fit any program objective. If the moments at beam midspan require additional reinforcing, it may be added. As the beam is designed to behave elastically, yielding will only take...
place in the ductile rod and not the beam itself. Consequently, in contrast to the prescriptions by ACI 318-14 high strength steel can be used for the longitudinal and for the shear reinforcement. The exact amount of shear reinforcement is again determined from the combination of $V_{ne}$ (Eq. 2) with the factored dead and live loads.

The load will be further transferred by bearing from the ductile rod to the column. Bearing stresses under the rod ends at the column-beam interface equilibrate respective shear loads (see Figure 3). As the shear load is only transferred at the zone of the frame in compression, and both the shim plates and the grout provide an additional confining pressure at this zone, the permissible bearing stress for confined concrete can be used appropriately6.

At an interior column connection with two adjacent beams using DDCs, bearing stresses at the middle of the column where both ductile rod plates meet have to be examined in more detail. Here, the ends are subjected to tensile load from the rod on one side and compressive load from the rod on the opposite side. As the ductile rod is designed to be the yielding element of the connection, the tensile load will eventually exceed the nominal tensile strength of the ductile rod $T_{yi}$ and overstrength factors need to be taken into account again. Consequently, the worst case bearing load that is imposed on the anchored end of one ductile rod will be $2\lambda_0 T_{yi}$. However, to determine the bearing strength of the connection, this load is too conservative because any overstrength compression load will be resisted by bearing on the face of the column. The resisting concrete in this part of the column is well confined, and the bearing area is smaller than the supporting surface at all sides. The design bearing strength may therefore be derived conservatively according to ACI 318-14 Section 14.5.6 as follows6:

$$2\cdot\phi_b\cdot(0.85\cdot f'_c\cdot A_l)$$  \hspace{1cm} (5)

The further shear load path inside the joint can be evaluated using strut and tie modeling. In traditional monolithic beam column joints, a primary diagonal strut links load nodes at both sides of the joints. Simultaneously, a second load path is generated by bond transfer mechanism and creates a substrut and tie load path. Once the primary diagonal strut reaches its maximum resistance and the secondary substrut mechanism cannot be activated anymore due to deteriorating bond stresses, a truss mechanism evolves and a much wider compression field is created. The compression field of the primary and secondary load path will ultimately break down once diagonal cracking becomes severe, and a pure truss mechanism will form8.

![Figure 3: DDC load path](image)
When using DDCs, it may be reasonably assumed that the tension load will flow directly to a secondary strut being developed starting from the ductile rod head anchored in the joint center. This has been verified by tests that are described in detail in chapter 2.3.8 of Reference 6. The compression load must be drawn back to the node that activates the primary strut, and thus, a primary diagonal strut forms. As the various struts are significantly stressed during cyclic displacement of the beam-column joint, confinement reinforcement needs to be provided to restrain crack growth7. The area of steel required can be determined with the total yield force at the joint \( T_y \) as follows:

\[
A_{sh} = \lambda_0 T_{yi} / (\phi_b F_{yt}) \tag{6}
\]

This tie area needs to be distributed over a distance of less than 18” [457 mm] above and below the ductile rod within the column, and the same tie configuration and spacing must be continued through the rest of the joint.

4 Connector variations

4.1 Column-foundation connection

In all descriptions above, reference is made to inner or outer beam-column connections in which ductile connectors may be applied. However, the application of the DDC is not just limited to this connection type and can also be effectively applied at column-footing connections. All design provisions and the load transfer described in chapter 2.2 and 3 remain the same, with the column replacing the beam and the footing replacing the column. The precast columns can be set over temporary guide studs, shims can be added to plumb the column at the appropriate height, and then the bolts can be tightened. Afterwards, the beams can be lifted and quickly connected to the columns by bolting, making the frame set up simple, effective and fast. The Wiltern Center Parking Structure in Los Angeles, the first DDC project in 1996, is one example where this method has been applied successfully9.

4.2 Super Hybrid System

Another innovative variation is the Super Hybrid System in which the standard DYWIDAG Ductile Connector as the main energy-dissipating element of the frame is combined with a post-tensioned strand in the center of the frame as the restoring force (see Figure 4 left). Just like for the standard DDC, precast components can simply be joined together by bolting. The post-tensioning required for the beam provides additional strength in the frame and may be added at any time. No grouting in the post-tensioning ducts, at the beam-column interface or the assembly pockets is necessary, which is a major advantage in comparison to purely hybrid systems. Furthermore, the addition of strands immensely improves the self-centering response as a restoring moment in the beam is provided. Consequently, this Super Hybrid System allows even higher achievable moment resistance with an excellent seismic performance. Instead of providing multiple ductile rods at a high seismic demand and increasing the overall beam size, the addition of an additional strand allows the beam size to be significantly reduced with sufficient moment strength. The total nominal strength at the connection may be determined as follows:

\[
M_n = N \cdot T_{yi} (d-d') + f_{pc} A_{pc} (h/2 - d') \tag{7}
\]
\[ V_{bn} = \frac{M_n}{l_{bc}} \]  

Where \( f_{pse} \) is the initial stress, \( A_{ps} \) the area of the provided strand, \( h \) the total beam height and \( l_{bc} \) the clear beam span.

The efficiency of this system has been proven by its application at the 39-story Paramount apartment building in San Francisco. It was the tallest precast pre-stressed concrete framed structure in the highest seismic zone 4 at the time of its construction and continues to be the highest building of this kind in the West Coast of the USA\(^{10}\).

### 4.3 Cast-in-place

The application of the DDC system is not simply limited to precast concrete structures. Its various benefits regarding seismic resistance and constructability also justify its usage in pure cast-in-place concrete structures or mixed cast-in-place and precast buildings in seismic zones. As in this case the bolting of the assembly becomes redundant, high strength bolts and the connector plate can be omitted, and the high strength threadbars can be threaded directly into the ductile rod that is embedded within the column (see Figure 4 right). The Medaillion building in Los Angeles and the Paramount building in San Francisco are only two examples of many successful applications in which the DDC connector has been used to join cast-in-place beams to precast columns.

![Figure 4: Super Hybrid System\(^8\) (left), Cast-in-place DDC System\(^8\) (right)](image)

### 5 Testing

#### 5.1 Need for test evaluation

In ACI 318, the maximum actual yield strength of longitudinal reinforcement in special moment frames in earthquake resistant structures is limited to 60 ksi [420 N/mm\(^2\)]. According to ACI 318-14 R18.2.6, the use of rebars with higher strength than those used by the design code will result in higher stresses (both shear and bond) when yield moments are developed and may lead to brittle failure. As explained in chapter 2.2, the DYWIDAG threadbars that are used as the main longitudinal reinforcement within the beam exceed this defined limit state. However, for general concrete structures, the code also states the following in section 18.2.1.7:
“A reinforced concrete structural system not satisfying this chapter shall be permitted if it is
demonstrated by experimental evidence and analysis that the proposed system will have strength and
toughness equal to or exceeding those provided by a comparable reinforced concrete structure
satisfying this chapter.”

Since its first conceptual development, both the standard DDC and the Super Hybrid System have
been tested in various subassembly and full-size-tests mainly at the University of California at San
Diego (UCSD), and the demanded superiority to monolithic structures has been proven every time.

5.2 Testing of the DYWIDAG Ductile Connector

After developing the concept of the DDC in the 1990s, a first test setup was generated in order to
verify the load transfer and the general connector. A prototypical beam and column subassembly
with a DDC connection and a code-compliant cast-in-place frame for reference were tested under
cyclic loading to failure using the loading sequence proposed by PRESSS. The hysteresis for both
systems is shown in Figure 5. It can be followed that despite having dissipated more energy per
cycle, the monolithic joint was not able to reach a drift of 4% and showed a considerable loss of
strength. This can be easily observed visually by the large structural damage that occurred at this test
frame. The ductile concrete frame, however, was able to withstand cycles up to a drift of 4.5% with
considerably less strength degradation and damage. Even after three cycles at a very large drift of
4%, the DDC system did not show any sign of large strength degradation. In summary, the total
displacement capacity of the tested conceptual ductile frame was significantly better than that of the
monolithic frame, and hence, it completely lies within the restrictions laid out by the ACI 318. The
test setup and its results are discussed in more detail in Reference 3.

![Hysteresis of DDC frame (left) and monolithic frame (right)](image)

Figure 5: Hysteresis of DDC frame (left) and monolithic frame (right)

The latest full-scale interior beam-column subassembly testing of the standard DDC was performed
at the University of California in San Diego in 2007. High strength columns with a total of 12
embedded ductile rods were precast and, after erection, cold joined with cast-in-place normal
strength concrete beams in both directions. As the beams were cast-in-place, no connector plate and
high strength bolts were required, and the DYWIDAG threadbars could be threaded directly into the
ductile rod. The loading of the subassembly was introduced through two 220 kips [979 kN] actuators
that were placed vertically. During the reversed cyclic loading, a constant axial compression of 150
kips [667 kN] was applied to the column. Displacement at the top of the column was restrained by a
single 500 kips [2224 kN] actuator, while the bottom of the column was restrained through a steel
pin\textsuperscript{11}.

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In the left hysteresis of Figure 6, the average shear force between the two vertical actuators is displayed against the average drift ratio at the beam-ends. The peak load that could be reached by the ductile frame was 176 kips [783 kN] in the push and 172 kips [765 kN] in the pull direction. Again, for this specimen, no signs of strength degradation could be observed, and the hysteretic response was generally very stable. The test was terminated with a maximum beam displacement of 8.5” [21.59 cm], which accounts for an average drift ratio of 7.1%. When looking at the energy dissipation, it could be noted that with an increasing drift ratio, the energy dissipated also increased. The maximum value of joint shear stress recorded was just 43% of the joint shear stress capacity per the method in ACI 352.

![Figure 6: Hysteresis of DDC (left) and Super Hybrid System (right)](image)

First yield strain of the frame was recorded in the ductile rod at a drift ratio of 5.35% before any yielding occurred at the beam threadbars. Consequently, the provided rod dictated the yielding of the whole specimen, and the desired behavior of the DYWIDAG Ductile Connector could again be verified. A more detailed description of the test setup and evaluation of the obtained results can be found in Reference 11.

## 5.3 Testing of the Super Hybrid System

Apart from the standard DDC system, a subassembly using the Super Hybrid System was tested in 2007 as well. Again, high strength precast columns were used, but with a total number of eight embedded ductile rods. Two different precast beam configurations were tested simultaneously. The beam at the left side used the standard DDC assembly with connection plate and high strength bolts and can most effectively be used in pretopped precast systems. The top threadbars of the right beam on the other hand, were threaded directly into the ductile rods and the connection plate and high strength bolts were only used at the beam bottom to simulate an appropriate system for topping slaps or cast-in-place floor systems. After the slabs had been constructed, 11 0.6” diameter post-tensioned un-grouted strands were installed to an initial stress of 190 ksi [1310 N/mm²], resulting in a total post-tensioning force of 450 kips [2002 kN].

The test setup and the load sequence of this subassembly was identical to the one described in the previous chapter for the standard DDC, and overall, the same observations could be made. Again, the average shear force versus the beam drift ratio displayed in Figure 6 (right), shows a fairly
symmetric hysteresis and a stable hysteretic response of the system. The peak load in the push direction was 170 kips [756 kN] and 141 kips [627 kN] in the pull direction. The observed asymmetric response is logical, as the compressive block regions were detailed slightly differently. The test was terminated at a maximum drift ratio of 7.06%, and no post-yield strength degradation was observed. The maximum joint stress was just 43% of the joint shear capacity of ACI 352.

The nominal moment capacity of the Super Hybrid specimen and the standard DDC specimen were similar despite using only 8 instead of 12 ductile rods. The added PT strand improved the self-centering response of the frame by providing a restoring moment in the beam, and the recorded energy dissipation of the frame significantly exceeds that provided by an ACI compliant system (see Figure 5). A more detailed evaluation of the obtained results can be found yet again in Reference 11.

Figure 7: Super Hybrid System after achievement of 5.25% drift

6 Conclusion

The development of the DYWIDAG Ductile Connector has proven once again that when using out-of-the box thinking, innovative solutions can be both economical and structurally superior to any traditionally evolved systems. Hence, creativity and the adaption of new ideas should be borne in mind by any structural engineer, especially in the software based traditional design process that dominates today.

When using the DYWIDAG Ductile Connector as the main frame-connecting system, any designer can unconditionally profit from the immense advantages of precasting and still obtain a system with excellent structural stability, even in the highest seismic zones. Both the standard DDC system and the Super Hybrid System have proven their superiority to monolithic structures in numerous tests and have shown that they can withstand story drifts of up to three times the limits specified in the code with considerably less overall damage. The developed ductile connector system has been effectively applied in various buildings in highly seismic, but also in non-seismic zones over the last thirty years. In all of these applications, the building owner and designer could profit from immense time and cost advantages.
7 Acknowledgement

All of the information presented in this paper is based on the far-reaching years of theoretical research and practical tests carried out and strongly supported by the inventor Prof. Dr. Englekirk up to today. His passionate conviction and vast, patient and constant explanations are highly appreciated.

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ANCHORAGE OF CLADDINGS WITH GROUTED ANCHORS

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ABSTRACT

Grouted anchors represent a traditional way especially of the anchorage of natural stone cladding to the concrete structure but are also used for the anchorage of cladding panels produced from manufactured stone. Then large holes are drilled in the structure, cleaned and the metal grout-in anchor is anchored with a specified cementitious grout. In Germany their use is covered in case of natural stone cladding in DIN 18516-3 and in case of manufactured stone claddings in DIN 18516-5 in the same manner.

In contrast to products to be designed according to EN 1992-4 whose suitability and use has to be stated in a European Technical Product Specification, cementitious grouted anchors are standardized products. Material and shape of the anchors as well as the grouts used for the anchorage and the installation terms are given in DIN 18516-3 and in DIN 18516-5. A total of four design methods for grouted anchors are offered. They are identical in both standards and differ in the complexity and the requirements on the properties of the grouts to be used.

Practical applications of grouted anchors will be shown and a general overview on the background of the regulations for grouted anchors and their design will be given.

1 Introduction

Metal grout-in anchors are used to fix natural stone cladding and cladding panels produced from manufactured stone to the concrete structure. They are installed into large boreholes and anchored with cementitious grout. In Germany their use is covered identically in case of natural stone cladding in DIN 18516-3 and in case of manufactured stone claddings in DIN 18516-5.

Grouted anchors are standardized products. Material and shape of the anchors as well as the grouts used for the anchorage and the installation terms are given in DIN 18516-3 and in DIN 18516-5. A total of four design methods for grouted anchors are offered. They differ in the complexity and the requirements on the properties of the grouts to be used.

In the following a general overview on the regulations for grouted anchors and their design is given.
2 Grout-in anchors

Grout-in anchors were developed as support anchors (Figures 1 and 2), which are able to resist loads acting in all directions, and restraint anchors (Figure 2) to resist wind loads (tension and compression) and restraints only. They are produced from stainless steel with at least resistance class III corresponding to the German Approval Z-30.3-64. The cladding panels are fixed to the projecting ends of the anchors by means of pins. Figure 3 shows an example.

Figure 1: Support anchor – flat steel anchor with twisted tail (40° to 90°) as anchorage element (Type 1), example\textsuperscript{1}

Figure 2: Support or restraint anchor – round steel with wave shaped tail as anchorage element, example\textsuperscript{2}
Grouted anchors are anchored in a big borehole or a recess filled with grout. This requires expensive hammer drilling machines and hammer drills which might significantly strain the installer’s physis. However, the big borehole diameter (up to $d_0 = 50$ mm) permits a good and simple adjustability based of the anchor in all directions. This means, that it is accepted that the anchor is not situated in the center of the hole. However, an adverse effect is that large borehole diameters cannot be drilled in highly reinforced structures without damaging or even cutting the structural reinforcement (Figure 4) and without reducing the bearing section of the concrete component. In addition, it has to be noted that the compressive strength of the grout is less than the compressive strength of the concrete, in general. These effects might yield unfavorable consequences for the bearing structure with regard to its load bearing capacity. Grouted anchors also represent thermal bridges which are to be considered in the energy-technical verifications.
3 Grout

The following types of grout are permitted for the anchorage of grout-in anchors:

a) Mortar for masonry class M10 according to EN 998-2,

b) Anchorage mortar according to EN 1504-6,

c) Factory production controlled dry mineral mortar with a characteristic compressive strength of at least 20 N/mm², fulfilling in minimum the requirements of mortar for masonry class M20 according to EN 998-2.

d) Cementitious grout according items a) to c) in connection with the DAfStb-guideline ‘Herstellung und Verwendung von Trockenbeton und Trockenmörtel’ (‘production and use of dry concrete and dry mortar’)⁸

e) Cementitious grout according to items a) to c), where the fitness for the intended use is stated in a European Technical Specification or where conformity is demonstrated for the particular case. The verification for the fitness of use is generated by means of a testing guideline analogous to ETAG 001, Part 5⁹ to determine the suitability and admissible service conditions of bonded fasteners.

Grouted anchors shall be installed only if the temperature of the concrete component serving as base material is T ≥ 5°C. The point in time of the first loading of the anchor depends on the temperature and the strength of the base material. The earliest point of loading is also governed by the product specific curing process and varies between one and 14 days.

4 Design

4.1 Basis

A total of four design methods, i.e. the methods 'A' to 'D', which differ in their complexity and the requirements on the grout to be used are available for the design of grouted anchors (Figure 5). Method 'A' is a simple approach which is in most cases sufficient. Method 'D' needs the most time and effort with the benefit of the most economical solution.

Independent of the selected method the installation and execution requirements stated in Ref.¹ and Ref.² apply and serve as basis for the design. Furthermore the following shall be observed:

- the limitations for the borehole diameter $d₀ (d₀ ≤ 50 \text{ mm})$,

- the embedment depth $h_{ef} (h_{ef} ≥ 80 \text{ mm or } h_{ef} ≥ 2d₀ + 10 \text{ mm respectively})$,

- the minimum values for edge distance $c$ ($c_{min} = 80 \text{ mm bzw. } 2d₀$) and spacing $s$ ($s_{min} = 100 \text{ mm or } 3d₀$, respectively) as well as

- the member thickness $h$, dependent on the type of grout-in anchor (support anchors: $h ≥ 150 \text{ mm}$ or $h ≥ h_{ef} + 2d₀$, respectively; restraint anchors: $h ≥ 120 \text{ mm or } h ≥ h_{ef} + 2d₀$, respectively)

In all cases the larger value governs.
The verification of the anchorage with grouts occurs analogously to the design procedure provided by EN 1992-4, where it has to be confirmed that the value of design action is smaller than the design value of resistance.

**Grout: Mortar acc. to DIN 1045-2**  
Characteristic resistance acc. to CEN/TS 1992-4, independent of the load direction  
\(\tau_{\text{Rk}} = 1.0 \text{ N/mm}^2\), failure mode not considered

**Mortar: prequalified System**  
Characteristic resistance acc. to CEN/TS 1992-4, dependent of the load direction  
\(\tau_{\text{Rk}} = 1.0 \text{ N/mm}^2\), failure mode considered

**Method ‘A’**  
Grout: Masonry mortar, Class MG III  
Characteristic resistance independent of the load direction  
\(\tau_{\text{Rk}} = 1.0 \text{ N/mm}^2\), failure mode not considered

**Method ‘B’**  
Grout: Masonry mortar, Class MG III  
Char. resistance acc. to CEN/TS 1992-4, dependent of the load direction  
\(\tau_{\text{Rk}} = 1.0 \text{ N/mm}^2\), failure mode considered

**Method ‘C’**  
Grout: Mortar acc. to DIN 1045-2  
Char. resistance acc. to CEN/TS 1992-4, independent of the load direction  
\(\tau_{\text{Rk}} = \text{variable, failure mode considered}\)

**Method ‘D’**  
Mortar: prequalified System  
Char. resistance acc. to CEN/TS 1992-4, dependent of the load direction  
\(\tau_{\text{Rk}} = \text{variable, failure mode considered}\)

**Figure 5:** Design methods for grouted anchors according to DIN 18516-3 and DIN 18516-5

The design of the anchorage shall be performed by experienced personnel, reviewable calculations and construction drawings taking into account the loads to be anchored shall be provided. In addition the transfer of the forces from the anchorages into the structure must be verified by computation.

Explicitly it is noted that the construction drawings should include the type of grouted anchor to be used, the drilling method, the borehole diameter and the borehole depth as well as the position of the anchor.

### 4.2 Method ‘A’

Method 'A' for the determination of the resistance of grouted anchors represents a simplified approach on the basis of the so-called Kappa method which was part of the German approvals for headed bolts and post-installed fasteners in the past. It consists of one characteristic resistance \(F_{\text{Rk}}\) for tension, combined tension and shear as well as shear loading and all possible failure modes. This characteristic resistance is calculated by multiplication of the effective surface area of the grout-in anchor with the characteristic bond resistance of the grout (Equation (1)). The embedment depth taken into account in the design shall not exceed 120 mm.

\[
F_{\text{Rk}} = A \cdot \tau_{\text{Rk}}
\]  
(1)

where

\(\tau_{\text{Rk}}\) characteristic bond strength
\[ A = U \cdot h_{ef} = \text{surface area of the anchor} \]

\[ U \] effective circumference of the anchor, see Ref.\(^1\) or Ref.\(^2\), respectively

\[ h_{ef} \] effective embedment depth of the anchor: \(80 \text{ mm} \leq h_{ef} \leq 120 \text{ mm}\)

The characteristic resistance of a grouted anchor influenced by a corner and/or an adjacent anchor with small spacing is:

\[ \text{red } F_{Rk} = \left( \frac{c_1}{c_{cr}} \right) \cdot \left( \frac{c_2}{c_{cr}} \right) \cdot \left( 1 + \frac{s}{s_{cr}} \right)/2 \cdot F_{Rk} \quad (2) \]

where

- \( F_{Rk} \) characteristic resistance of a grouted anchor not influenced by edges or adjacent anchors according to Eq. (1)
- \( c_1 \) distance to edge 1 (\(c_{\min} \leq c_1 \leq c_{cr}\))
- \( c_2 \) distance to edge 2 (\(c_{\min} \leq c_2 \leq c_{cr}\))
- \( s \) spacing in case of two neighboring anchors (\(s_{\min} \leq s \leq s_{cr}\))
- \( c_{cr} = 120 \text{ mm} \)
- \( s_{cr} = 240 \text{ mm} \)

Method 'A' applies exclusively to anchorages in normal weight concrete of at least concrete strength class C12/15. It is to be noted that as basis for the determination of the anchor load-carrying capacity in C12/15 and C16/20 results of tension tests with grouted anchors performed at the relevant structure must be available. For base material of the concrete compressive strength class C20/25 the characteristic value of the bond strength of \(\tau_{Rk} = 1.0 \text{ N/mm}^2\) can be used for the design. If it is verified that the concrete remains uncracked in the anchorage zone for the whole service life the bond strength of \(\tau_{Rk} = 1.4 \text{ N/mm}^2\) can be taken.

The threshold values of the characteristic bond strength of \(\tau_{Rk} = 1.0 \text{ N/mm}^2\) or \(1.4 \text{ N/mm}^2\), respectively, were derived from appropriately 400 tests with different types of grouted anchors performed by the University of Applied Sciences of Cologne. The tests were carried out at a building where the cladding was removed for retrofitting measures. The test results are plotted in Figure 6 together with results from on-site and ‘old’ and ‘new’ laboratory testing. Detailed information is given in Ref.\(^{11}\).

Figure 6 also shows the statistical analysis of the individual test series. The thick black line represents the average value of all test series, the thick dashed line the corresponding 5%-fractile at a 75% confidence level, the thin dashed lines the 5%-fractiles of the test locations and the thin black polygonal line the 5%-fractile of the individual test series. Based on these evaluations the applicable bond strength of grouted anchors installed to base material of concrete strength classes C12/15 and C16/20 was limited to \(\tau_{Rk} = 1.0 \text{ N/mm}^2\) or \(1.4 \text{ N/mm}^2\), even if the results of the building site tests performed at arbitrary locations would have permitted higher values. This is due to the fact, that the grouted anchors which are tested in the structure are usually installed with raised care (Figure 7a)) and without the unfavorable influence of already mounted cladding panels and existing thermal insulation (Figure 7b)) representing barriers for the installation. Hence, in the tests at the structure
usually do not consider the complicated placing of the right amount of grout into the borehole, the insertion of the grout-in anchor and the subsequent compaction of the grout. In addition, it shall be noted that the removed cladding was originally installed by a company with a good reputation.

Figure 6: Results of grouted anchor testing

Figure 7: Installation conditions
a) grouted anchor before in-situ test b) installation on site

To facilitate the determination of the resistance of grouted anchors DIN 18516 3, Table 4, and DIN 18516-5, Table 6 give characteristic resistances of current sizes of support and restraint anchors.
made of round and flat steel subjected to tension, shear and combined tension and shear loads and located in cracked concrete of at least class C20/25. They range between $F_{Rk} = 1.5\,\text{kN}$ for a round steel anchor $\varnothing 6\,\text{mm}$ with an anchorage depth $h_{ef} = 80\,\text{mm}$ and $F_{Rk} = 10.8\,\text{kN}$ for a flat steel anchor with a circumference of 90 mm and an anchorage depth $h_{ef} = 120\,\text{mm}$.

4.3 Method ‘B’

Method ‘B’ corresponds to the design procedures of CEN/TS 1992-4-5\(^{12}\) and DIN EN 1992-4\(^{3}\) for the determination of the resistance of bonded fasteners. Thus the verification includes the actual direction of the action and the decisive failure mode. This yields substantially higher characteristic resistance in comparison to the values determined according to Method ‘A’.

The applicability of the design procedure for post-installed bonded fasteners requires the determination of grout-in anchor specific input parameters. These include the effective circumference of the anchor, because grout-in anchors can have arbitrary cross sections, as well as the effective diameter resisting the shear load, if the grout-in anchor does not have a round cross section. Furthermore the verification for concrete pry-out failure is adjusted based on the specific application.

As in Method ‘A’ the usable bond stresses are limited to the $\tau_{Rk} = 1.0\,\text{N/mm}^2$ or $1.4\,\text{N/mm}^2$. In case that the higher bond strengths of dry mortars according to the DAfStb-guideline\(^{8}\) should be used Method ‘C’ can be applied, in case of grouts with verified performance according to a European Technical Specification Method ‘D’.

4.4 Method ‘C’

Method ‘C’ agrees basically with Method ‘B’. It applies explicitly to anchorages in concrete of at least class C20/25 in connection with the DAfStb-guideline ‘Herstellung und Verwendung von Trockenbeton und Trockenmörtel’ (‘production and use of dry concrete and dry mortar’)\(^{8}\). The maximum grout compressive strength class permitted to be used in the design equations is C40/50. Then the characteristic bond strength for the usual case of cracked concrete serving as base material is:

$$\tau_{Rk} = 0.22\, f_{ck}^{2/3}\,\text{[N/mm}^2\text{]}$$

(3)

If it is verified that the concrete remains uncracked in the anchorage zone for the whole service life, the bond strength calculated according to Equation (3) may be raised by the factor 1.4.

4.5 Method ‘D’

Method ‘D’ corresponds basically to Method ‘C’. Deviating from Method ‘C’, Method ‘D’ applies only to the anchorage in normal weight concrete with a grout where its fitness of use is demonstrated by a European Technical Specification or where conformity is demonstrated for the particular application.

The verification for the fitness of use is generated by means of a testing guideline analogous to ETAG 001, Part 5\(^{9}\) to determine the suitability and admissible service conditions of bonded fasteners in concrete and stated in a corresponding document. This document does not exist up to now, but is
intended to provide product specific requirements for the installation, the application conditions and
design values such as the characteristic bond strength which is not limited.

Currently there are still no cementitious grouts on the market which permit the design of grouted
anchors according Method 'D'.

5 Summary

Grouted anchors represent a workmanship especially of the anchorage of natural or manufactured
stone cladding to the concrete structure. To fulfill the requirements of practice different types of
anchors and mortars can be used. Their application and possible problems developing from
improper installation and interaction with the concrete structure are described as well as how these
effects are considered in the actual design procedures of DIN 18516-3 and DIN 18516-5.

For the design of the anchorage by means of grouted anchors in Ref.¹ and Ref.² procedures are given
which take into account the current practice. For the simple way of design which covers the most
part of the application cases with conventional cementitious grouts Method 'A' can be used. Special
cases can be solved with the same type of grouts by applying Method 'B'. If even higher resistances
are required, special grouts are to be used. The design of anchorages with these special grouts can
occur either after Method ‘C’ or Method ‘D’.

Furthermore it is advantageous that the design rules for grouted anchors are based on procedures
which are used in fastening technology with inserts and post-installed fasteners already for a long
time.

Thus the design rules for grouted anchors given in Ref.¹ and Ref.² take into account all demands
from the practical application.

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RESPONSE OF FLANGE-TO-FLANGE DOUBLE TEE CONNECTIONS
SUBJECT TO TRANSVERSE LOADING

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ABSTRACT

In the United States, the most common component in parking garage structures is the double tee. A typical beam has a top flange that is 100 mm thick. Steel embedments, or connectors, are cast at intervals into the edges of the top flanges of the double tees. The double tees are laid side by side and joined together by fillet welding plates, called jumper plates, between the connectors. The purpose of these connectors is to provide a uniform driving surface and to create a rigid diaphragm necessary to resist lateral loads.

As vehicular wheels pass over the joints between double tee beams, a load is imposed on the connectors, and likewise, the jumper plate welds are stressed about their weak axes.

Concerns¹ have been raised regarding cyclic loading of such connectors and the potential for fatigue cracking of the welds. Several field studies have indicated that, although uncommon, fatigue cracks do occur, sometimes resulting in complete weld separation and at multiple connectors.

A study was undertaken by the Precast/prestressed Concrete Institute²,³ to better understand the problem and to determine the magnitude of stress ranges typically experienced by the jumper plate welds. The study involved both field and laboratory testing and finite element modeling. The finite element models were calibrated to the test data, then used to study the effects of various parameters such as weld sizes, jumper plate dimensions, and double tee dimensions. Analytical fatigue models were also studied.

1 Introduction and Background

Precast/prestressed concrete double tee beams are the most common type of floor system in parking decks built in the United States (Figure 1). A typical double tee is 3.0 to 4.6 m wide, 18 m long, and 760 mm deep (Figure 2).
The top flange is typically about 100 mm thick. In each double tee, the two stems are positioned about 1.8 m apart and the flanges overhang the stems on each side by about 0.9 m. In a typical garage, the double tees are laid side by side with a 6 to 21 mm gap between flange edges. The flange edges are typically joined at 0.9 m to 2.4 m spacings by “flange to flange connectors” (Figures 3 and 4). The flange to flange connectors serve to integrate the double tees as a floor diaphragm to resist lateral seismic and wind loads imposed on the parking structure. They also aid in alignment, resist thermal stresses, and resist vertical wheel loads. End of span connections provide diaphragm chord reinforcement for structures subject to seismic loads (Figure 3).

Double tee flange connections typically consist of a stamped steel plate cast into the edge of a concrete double tee flange. The stamped steel plate consists of a face plate and two wings. The wings are embedded in the concrete while the face plate is exposed along the flange edge. Once the double tees are set, a steel plate or rod is welded to adjacent face plates and welded to each one using a fillet weld. This steel plate or rod is typically referred to as a jumper plate, an erection plate, or a slug. Thus, a typical flange connection consists of two embedded connectors joined by a jumper plate (or round slug), connected with two field-applied welds, as illustrated in Figure 4. Embedded connectors are available in several different styles from various manufacturers. Three commercially available connectors in the US are shown in Figure 5.
2 Behavior of Flange to Flange Connections

This paper concerns itself primarily with the transverse (vertical) loading of flange to flange connectors. Frequent transverse vertical loads are imposed by automobiles. Forces from automobile wheel loads cause the loaded double tee to deflect. A portion of the vehicular load travels along the double tee beam to its bearings. The remainder of the load is transferred to the adjacent double tees through the flange to flange connectors. According to Reference 4, the average sedan and SUV in the US weighs about 18KN, including the driver. From this, we deduce that the average front wheel load is about 5 KN.

As the vehicle travels across the double tee beam to the adjacent beam, the loaded flange bends as a propped cantilever, transmitting vertical shear and thrust across the connections and imposing shear, axial force, and moment on the weld. When the vehicle passes to the adjacent double tee, the shear direction reverses as illustrated in Figure 6 and the weld stresses also reverse.
3 Number of Cycles

The traffic in a parking structure varies considerably based on the customers being served. For example, a residential garage would likely be full seven days per week while a garage that services an office building will be full five days per week. A garage that serves a retail shopping mall may only be fully occupied a fraction of the year. Residential and office building garages may see each occupant exit and enter only once per day while garages at shopping centers may see numerous entrances and exits per day for each parking stall. Consequently, over a 30-year life, the cyclic loading on the flange connections can vary considerably based on the usage of the structure and the location of connection within the garage. Depending on function of the garage and the location of the connection within the garage, estimates of cycles imposed on the flange to flange connections range from 100,000 cycles to 20 million cycles.

4 Developmental Testing of Components

The three predominant US manufacturers have performed developmental tests of their products, including in-plane shear, in-plane tension and transverse shear tests. In all cases, tests were performed on specimens having a connector embedded in one side of a concrete panel, a plate welded to the connector face, and a concentrated load applied on the plate. In no case did any manufacturer weld two adjacent concrete panels together. In no case was cyclic or fatigue loading considered in any of the available published data.

In a similar fashion, connectors from all three manufacturers were recently tested at Lehigh University. The tests involved mild carbon steel and stainless steel connectors statically loaded upward or downward. The test setup is indicated in Figures 10 and 11, and the load was applied 25 mm from the face of the connector. The Lehigh results generally matched published manufacturer data. Parameters that appeared to influence the results were the loading direction, the manufacturer, and the type of steel (Figure 12), however, variations in results appear to stem from small testing variations rather than any intrinsic differences between the connectors but they are instructive because the variation in test results was quite large.

![Figure 10: Typical “jumper plate”](image1)

![Figure 11: Assembly ready to test](image2)
In summary, connectors stiffness averaged 7 KN/mm, first yielding averaged 25 KN, and ultimate strength averaged 30 KN (Figure 12).

One developmental test series was performed by North Carolina State University for a private client. The results of that testing were reported in Reference 3. In that project, two double tee beams were connected along their flanges and then subject to concentrated vertical loads until failure. For situations where loads were imposed adjacent to a connection, the ultimate connector strength averaged 59 KN. When the loads were centered between connectors, the ultimate strengths averaged 64 KN.

5 Full Scale Tests at Tindall Corporation in Petersburg, VA

Full-scale experiments on three adjacent connected double tees were performed to measure member strains and deformations for use in refining analytical models. The experiment was based on field-realistic 3.7 m wide, 18 m long, and 760 mm deep double tees joined together with one connection type along one joint and another along a second joint. Instrumentation included measurement of applied load, connection strains, and member deflections. A maximum selected test load of 13.3 KN was applied to points along the flanges edges. A concentrated load of 13.3 KN matches the code required car-jacking load but is substantially higher than typical wheel loads. The goal of the full-scale experimentation was to measure flange deflections and connection strains under various loading conditions in an effort to refine finite element analyses and to verify connection behavior assumptions.

An overview of the test setup is presented in Figure 13. An instrumented connection is presented in Figure 14.
Typical measured strains are presented in Figure 15. Space limits prevent including all measured data. A full test report is presented in Reference 8.

**Manufacturer 1**

**Manufacturer 2**

Figure 15: Weld Strains at Connections Located 8.4 m from one end. High compressive strains correspond to the weld on the wheel loaded side of the joint. Note almost total absence of tensile strains in these connections, but tensile strains of up to 150 microstrain were measured elsewhere.
6 Finite Element Analyses

Many types of simple and complex finite element analyses were conducted as part of this research project. Space constraints dictates that we present only a small subset of the work here, but a more complete covering of the material can be found in Reference 9. The final baseline model of the three double tee beams and their connections was developed and analyzed in ABAQUS\(^1\). Altogether, 2.7 million elements were used to properly define the test beams and connection assemblies. The model was developed assuming that one edge of the face plate would bear against the face of the concrete flange. Contact elements were used to form that model. Similarly, contact elements were used to define the interaction of the jumper plate lower edge and the face plates of the connectors. On the wheel side of the joint, the jumper plate bottom edge is allowed to pull away from the face plate. On the opposite side the jumper plate lower edge bears against the face plate. Hence, the moment couple on the “opening” side of the joint is confined within the weld height (approximately 4 mm for a 6 mm weld) while the couple on the opposite side extends from the bottom of the jumper plate to the top of the weld (about 15 mm for a 6 mm weld plus a 10 mm thick jumper plate). If the moment applied to both sides is the same, the compressive stress on the “opening” side of the joint would be expected to be about four times the tensile stress on the opposite side of the joint. Comparison of the model with test results indicates that in almost all cases the jumper plate bears against the face plate as predicted by the model. In some cases of testing, the jumper plate bearing did not occur and the measured weld strains were much higher.

Differential displacement across the joints matched very well with the measured behavior as indicated in Figure 16. At a concentrated load of 6.7 KN adjacent to a connector a differential vertical deflection of 0.12 mm is predicted.

![Figure 16: Differential displacement across DT flange joint, FEA vs. test](image)

The main analysis result of interest is the stress located at the root of the weld. This is an impossible location to measure, so comparisons are only possible at the faces of the welds. Weld stresses were found by analysis to vary with location along the sloped surface as well as along the length. Ranges of weld stresses predicted by the model at the approximate locations of the strain gages matched reasonably well.

With a working model that correlates reasonably well to test data, connection component dimensions could be varied and the effect that such variations have on the weld stresses could be evaluated. In Figure 17, the results of analyses are shown corresponding to five different weld sizes ranging from 5
mm to 13 mm. A plot of average compressive weld stress versus the inverse square of weld size shows a linear relationship. Also varied were jumper plate widths and thicknesses. Plate widths studied were 19 mm, 25 mm, and 44 mm. Plate thicknesses studied were 6 mm and 10 mm. The variation in average weld compressive stress with plate width is presented in Figure 18. As is illustrated in the figure, average weld stresses vary linearly with plate width. Also shown in both Figures 17 and 18 are the shear and axial forces transferred across the jumper plate. The vertical shear forces are relatively constant 2.22 KN +/- 10% (i.e. approximately one third the applied load) regardless of plate or weld dimensions.

A comparison of variations in plate and weld dimensions and their effects on the weld stress ranges are summarized in Table 1. The predicted weld stress ranges for Manufacturer 2 were slightly higher than for Manufacturer 1. Varying the plate thickness was shown to have a minor influence on weld stresses.
Within the range of variables studied, weld face average stress ranges in psi were found to be inversely proportional to the weld size squared and directly proportional to the plate width.

### Table 1: Comparison of stress ranges predicted by FEA for varying sizes of plates and welds

<table>
<thead>
<tr>
<th>Case #</th>
<th>Weld size [mm]</th>
<th>Plate thickness [mm]</th>
<th>Plate width [mm]</th>
<th>Manufacturer 1, Stress range [MPa]</th>
<th>Manufacturer 2, Stress range [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>7.9</td>
<td>9.7</td>
<td>25.4</td>
<td>53</td>
<td>68</td>
</tr>
<tr>
<td>1</td>
<td>7.9</td>
<td>6.4</td>
<td>25.4</td>
<td>57</td>
<td>67</td>
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<td>5</td>
<td>12.7</td>
<td>9.7</td>
<td>25.4</td>
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<td>9.7</td>
<td>25.4</td>
<td>127</td>
<td>138</td>
</tr>
</tbody>
</table>

### 7 Analytical Prediction of Fatigue Strength

The authors found nothing in the literature that provided fatigue test data for connections of this type. The closest we found was research by Sorensen et al.\textsuperscript{11} which examined the performance of fillet welds that join lateral tubular members to column legs in offshore oil drill structures. Sorensen et al.\textsuperscript{11} devised a method to determine the critical stress at weld roots then estimate from that the fatigue life expectancy at any given stress. Sorensen et al.\textsuperscript{11} then tested the welds in the laboratory to confirm their analytical approach. Two fatigue stress parameters named SCF4 and SCF5 were developed in their study for use in determination of fatigue life. The SCF4 stress parameter is based on “linear extrapolation of the principal stress from two points in the fillet throat section to the weld root.” The SCF4 stress is obtained by extrapolating the maximum principal stresses at the quarter points on the weld throat section to the weld root. The SCF5 provides a better fit and is “based on linear extrapolation of the six basic stress components in the fillet throat section to the weld root.”

Using this approach an SN curve for SCF4 was developed and is reproduced in Figure 19. The mean S-N curve is given by the formula:

\[
\log N = 12.08 - 3 \log(\Delta \sigma)
\]

Where:

- \( N = \text{Number of cycles to failure} < 5,000,000 \)
- \( \Delta \sigma = \text{Stress Range at weld root using SCF4 stress parameter (MPa)} \)

The SCF4/SCF5 approach was developed specifically for details commonly used in the offshore structure industry. The detail differs from that of flange to flange connections in that the base plate to which the weld is attached is much more rigid than that of a typical manufactured connection faceplate. With a rigid boundary condition the stress at the root of the weld would be constant along the length of the weld. This is much more severe than the cases observed in the previously shown analyses where the weld has a high stress at the ends and a relatively lower value over the majority of
the length. Note that the lowest stress range sufficient to propagate a fatigue crack in Reference 11 (see Figure 19) was about 100 MPa.

Figure 19: Weld detail (Figure 12 - from Sorensen et al.11) and SCF4 Stress Parameter SN Curve (Figure 14 - from Sorensen et al.11)

8 Conclusions and Future Work

A comprehensive examination of flange to flange connections used for double tee connections in precast/prestressed concrete parking decks was conducted. The goal was to identify if the welded connections are susceptible to fatigue induced failures. The study included an informal survey of the industry, field measurements of differential connection deflection in existing parking structures, testing of commonly available connections, testing of a full scale prototype deck, and extensive numerical modeling of the response. The research results indicate the following:

- Informal polls of producers who had been fabricating and erecting double tee parking garages for decades indicated that weld fractures were uncommon and when they occurred were usually associated with atypical joint conditions or lack of maintenance. Problems with flange to flange joints reportedly do occur on occasion, but are typically due to atypical joint conditions or a lack of maintenance.

- An evaluation of typical parking structures indicates that connection cycles could exceed 20 million cycles in 30 years but only for a select number of garages and selected locations within those garages. The majority of connections would be subject to much lower number of cycles. Data available in the literature suggests that fatigue behavior need only be studied up to about 5,000,000 cycles.

- Observation of field measurements on existing parking structures indicate that differential displacements across the joints are on the order of 0.1 mm to 0.4 mm for a typical vehicle load.

- Connection stiffness in the vertical direction varies based on the connection configuration. Finite element analysis and experimental testing of conventional connections indicates that the out of plane stiffness typically ranges from 17.5 to 35 KN/mm.

- The FEA results show that the average stress levels in the fillet weld vary linearly to the inverse of the square of the weld size and also vary linearly with the jumper plate width.
One case study was presented in Reference 9 where fatigue cracks were found in a parking structure jumper plate welds. Measurements were taken of failed connections in that structure indicated that the weld was undersized and that the jumper plate was positioned very high. The jumper plates were also wider than is typical at other garages. Using the relationships developed in this study, the case study welds would be expected to be exposed to stresses four times greater than a standard connection.

The research results clearly indicate that the force transfer across a flange to flange connection as a result of vertical loads on the floor is complex. Stresses can be high in some cases but the occurrence of fatigue induced cracking is limited to a few instances. These cases, if they were to occur, would manifest at the middle of the double tee span where vehicles are most likely to pass. As shown in the study, the vehicle loads are resisted primarily by local flange flexure and the connections nearest the load point. Consequently, in the rare case that a fatigue induced failure were to occur it is unlikely that failures would propagate along the joint to other connectors outside the drive path. Furthermore, the integrity of the floor system to lateral earthquake and wind loads would be minimally impacted in the rare case of a fatigue induced failure. As shown, possible failures would be at the midspan leaving the connections at the ends of the double tee, especially the chord connection, intact to resist the required in plane shear and tension/compression forces. In other words, precast double tee parking structures are indeed safe against fatigue induced failure due to seismic events.

References

1. ABAQUS (2016) 'ABAQUS Documentation', Dassault Systèmes, Providence, RI, USA.


COMPARISON OF THE CYCLIC SHEAR BEHAVIOUR OF ROUGHENED REINFORCED INTERFACES OF LIGHTWEIGHT AND NORMALWEIGHT CONCRETES

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ABSTRACT

In repair or strengthening of existing Reinforced Concrete (RC)/ Lightweight Reinforced Concrete (LRC) structures, the transfer of shear from the existing element to new/added layers of concrete (and vice versa) is a prerequisite for the composite function of the structural member. In case of seismic actions, the behaviour of interfaces may be critical for the overall behaviour of the structure. It is noted that, due to the fact that interfaces in lightweight concrete are less rough, their behaviour is expected to be more critical for the overall behaviour of the structure, than in case of normal concrete.

Within a research programme carried out at NTUA, the experimental data available in the Literature regarding the behaviour of lightweight interfaces were collected to a database and evaluated. The literature survey has shown that the available test data on the cyclic behaviour of lightweight concrete interfaces were rather limited. Thus, a testing programme was carried out, including 6 tests adequately designed to simulate interfaces between old and new concrete, whereas the tests data (from the Literature and those obtained by the authors) were re-evaluated and used as a basis for the assessment of a physical model9,16 available in the Literature18.

The model takes into account both friction and dowel action along the interface, as well as their interaction. Furthermore, the contribution of each mechanism is predicted as a function of the displacement imposed to the interface. The bearing capacity of the interface is calculated for monotonic and for cyclic actions. The ability of the model to predict the experimental data is checked and confirmed through its application to the tests included in the database.

1 Introduction

The lightweight concrete, namely a concrete in which the aggregates are partly or entirely substituted by lightweight aggregates (Pumice stone), is a material with significant differences compared to the normalweight concrete. Laboratory tests have shown that the shear behaviour of the lightweight concrete is inferior compared to the behaviour of normalweight concrete: It is more brittle, of lower tensile strength, whereas (due to the lower strength of lightweight aggregates) the cracks cross the cement paste as well as the aggregates (EuroLightCon, 1988) resulting to less rough interfaces.
In several repair/strengthening techniques layers of RC are added to existing members (e.g. when an RC column is jacketed, when the height of a beam is increased, etc.). Thus, interfaces between existing and new concrete are formed, frequently crossed by metallic anchors. In case of interventions, the use of lightweight concrete may be preferable, given that its limited weight may allow its use where the normalweight concrete cannot be applied. The use of lightweight concrete may present advantages in other applications too, e.g. in prefabrication, given that its reduced weight allows for easier transportation and increase of the dimensions of the single elements. In this case, interfaces between precast elements to be connected in situ need to be designed.

The behavior of structural elements in the aforementioned applications, as well as in cold joints, along cracks within the element, etc., depends on the behaviour of RC interfaces. Therefore, the study of the behaviour of interfaces (in terms of both mobilized resistance and respective deformations) is a prerequisite for a rational design of LRC and RC structures.

2 Literature Survey

The results of numerous tests on (plain or reinforced) concrete interfaces are reported in the Literature. Tests simulate various cases of interfaces, such as construction joints, connections between precast elements, natural cracks, etc. In most of the tests, interfaces were subjected to monotonically increasing load up to failure. Data on the behaviour of reinforced interfaces simulating the interfaces between old and new concrete in repaired/strengthened elements, subjected to cyclic shear slip are rather scarce.

Shear is transferred along the interfaces thanks to the mobilization of two mechanisms, namely, concrete to concrete friction and dowel action of the bars crossing the interface. The two main shear transfer mechanisms were investigated mainly under monotonic actions, either separately or jointly (therefore, their interaction was also investigated). In most of experimental programs, the shear behaviour of interfaces was investigated using “push-off” type specimens. It is to be noted that, surprisingly enough, the available experimental results on the cyclic behavior of the shear transfer mechanisms date back to the 70’s and 80’s. Repeated or cyclic shear imposed to interfaces in tests until the 80’s are reported in detail elsewhere. In most cases, load-controlled tests were carried out and, hence, cycling was limited to shear forces smaller than the maximum resistance of interfaces. Therefore, no data are available regarding the post-peak behaviour of the mechanisms.

As far as interfaces in lightweight concrete are concerned, tests were performed exclusively under monotonic loading. The first published results regard four different lightweight concrete mixes, three of them produced using expanded Clay aggregates (LECA). More recent results are published by Papanicolaou et al., 2002, who have studied the behaviour of interfaces between lightweight and normalweight, high performance concrete, in various structural elements, typically of the sandwich type. These elements consist of relatively thin faces of high strength-fibre reinforced concrete (generally high performance concrete) and a lightweight aggregate concrete core. Scott, 2010, has studied the shear behaviour of lightweight concrete interfaces, simulating the joints in precast concrete bridges. Shaw et al., 2014 have studied experimentally the behaviour of reinforced lightweight concrete interfaces, between concretes cast at different times. The research has been
extended by Sneed et al., 2016, who have experimentally studied 52 push-off specimens to investigate the direct shear transfer of different types of concretes with different interface conditions. The aforementioned tests have shown that the shear resistance of LRC interfaces is lower than the resistance of RC interfaces. This behaviour can be attributed to the smoother form of the interfaces in LRC, where the cracks cross also the aggregates, as well as to the lower tensile strength of the material. Even though the cyclic shear behaviour of RC interfaces was experimentally and analytically studied at the Laboratory of Reinforced Concrete, NTUA within a series of research programs, experimental results regarding the cyclic behaviour of lightweight concrete interfaces were not available so far. Thus, a series of tests was carried out (6 tests adequately designed to simulate interfaces between lightweight concretes cast at different times), whereas the tests data (from the Literature and those obtained in NTUA) were re-evaluated and used as a basis for the evaluation of a physical model, proposed by the first Author.

3 Experimental Program

3.1 Specimens and Investigated Parameters

The scope of the testing program presented herein, part of a broader project investigating various aspects of the behavior of structural lightweight concrete, was to study the cyclic behaviour of lightweight concrete interfaces. Due to the inevitably limited number of tests (Table 1), the parameters to be investigated were selected so that they would allow for direct comparison with the already available experimental data. Even though two bar diameters were tested, namely 8mm and 12mm, in the present paper only the results regarding the 12mm diameter bars are presented, given that relevant test data (from RC interfaces already tested at NTUA) are available for direct comparison. Further parameters (compressive strength of the concrete in the two parts of the specimens, roughness of the interface, loading history, etc.) and other combinations of parameters are still to be examined. The following parameters are examined in the present paper:

a) **Compressive strength of existing and added concrete**: The compressive strength of the concrete was not a parameter within the present testing program. The compressive strength of both the existing and the added concrete was equal to 34MPa. Pumice aggregates were used for the construction of the specimens. It is noted, that in two of the specimens, steel fibres are added to the concrete, in order to study their effect on the behaviour of the interface.

b) **Embedment length of bars crossing the interface**: Specimens with limited embedment length of bars simulate the (quite common) case of repair and/or strengthening techniques, where the available thickness of the existing and/or the added concrete layer does not allow for sufficient anchorage of the bars across the interface. The embedment depth of the reinforcement normalized to bar diameter was equal to 12.5 or 20.

c) **Normal stress on the interface**: The favourable effect of the normal stress on the interface has been apparent during previous research work. Thus, in one specimen, normal compressive stress equal to 1.0MPa was applied to the interface.
Table 1: Main characteristics of specimens and experimental values of maximum shear resistance of LRC and relevant RC interfaces, reinforced with 12mm diameter rebars

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Number and diameter of bars/Reinforcement ratio/Embedment depth normalized to bar diameter</th>
<th>$\tau_{u,\text{exp}}$ (N/mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1L</td>
<td>R-34L/E/20/0.1 LC/3(\Phi12/0.0068/20)</td>
<td>2.92</td>
</tr>
<tr>
<td>2L</td>
<td>NR-34L/E/20/0.1 LC/3(\Phi12/0.0068/20)</td>
<td>3.91</td>
</tr>
<tr>
<td>3L</td>
<td>R-34L/E/12/0.1 LC/3(\Phi12/0.0068/12.5)</td>
<td>2.87</td>
</tr>
<tr>
<td>4L</td>
<td>R-34L/E/20/0.1 LC, fibers/3(\Phi12/0.0068/20)</td>
<td>5.74</td>
</tr>
<tr>
<td>1N</td>
<td>R-31/E/20/0.1 3(\Phi12/0.0068/20)</td>
<td>3.68</td>
</tr>
<tr>
<td>2N</td>
<td>NR-25/E/20/0.1 3(\Phi12/0.0068/20)</td>
<td>4.50</td>
</tr>
<tr>
<td>3N</td>
<td>R-30f/E/20/0.1 Fibers/3(\Phi12/0.0068/20)</td>
<td>3.99</td>
</tr>
</tbody>
</table>

1. Designation of specimens: R: rough interface, N: Normal force on the interface (equivalent to initial uniform compressive stress of 1.0 MPa) The first number indicates the compressive strength of the concrete (in MPa), L: lightweight concrete, f: (steel) fiber reinforced concrete. E: specimens with three bars 12 mm in diameter. The second number indicates the embedment depth normalized to the bar diameter. The third number indicates the magnitude of the cyclic shear slip imposed during the first cycle (in mm).

Figure 1: Geometry of the specimens with 8mm or 12mm bars (the reinforcing bars of the interface are not shown here).

The specimens (Figure 1) are of the same form as the ones used during previous testing campaigns at NTUA$^9$. The interface was 500mm long and 100mm wide. The reinforcing bars of 12mm were positioned at mid-width of the interface. The two concrete blocks forming each specimen were adequately reinforced aiming to avoid premature damage of the specimen outside the interface.

The specimens consist in two reinforced concrete blocks, cast separately into metal moulds, approximately 28 days one after the other. The reinforcing bars were positioned in the first concrete block before casting of the concrete and they were protruding to a predetermined length, so that the
bond with the second block was also ensured. The present research comprises test results for four (4) specimens with three bars of 12mm diameter crossing the interface, while all the experimental results regarding LRC interfaces are presented elsewhere\textsuperscript{15,17}. After concreting the first block, simulating the existing concrete, the interface was artificially roughened (chipped) using a pickaxe. A simple technique was followed for roughening the interface, as it is impractical to apply sand- or water-blasting on site.

In the specimens presented herein, the amplitude of the cyclically imposed shear slips, during the first cycle was not an investigated parameter. Three full reversals at slip of $\pm 0.10$mm were imposed. Subsequently, sets of three reversals at larger shear slip values were imposed, until the force response degradation became significant, or until the cracks hindered the continuation of testing.

3.2 Test set-up

Figure 2 shows the test setup, described in details elsewhere\textsuperscript{9,10,11}. The aim of the test set-up is to keep the specimen “S” attached to the actuator “A” in such a position that the axis of the piston coincides with the interface. The actuator “A” (MTS double ended hydraulic Actuator) is connected to the “old” part of the specimen “S”, which is the one moving, while the “new” part is kept in position using a couple of steel plates “P”. The test set-up allows for normal compressive stress to be applied to the interface by means of additional steel rods “r” and actuator “a” (max. capacity=100kN). Shear slips were imposed to the interface by the actuator at low speed.

Figure 2c shows the instrumentation: The shear slip along the interface was measured by means of four LVDTs (channels 5 to 8) on both faces of the specimen, along with the force response of the interface, whereas, four LVDTs (channels 1 to 4), placed perpendicular to the interface, measured the width of the crack at the interface level. Electrical strain gauges (glued on steel bars crossing the interface before concreting, at a distance of approximately 10mm to 30mm) were used to measure the strains developed in the bars in the course of the test.

![Figure 2: Test setup](image)

(a) Sketch of the test set up applicable to specimens without normal compressive stress on the interface, (b) Photo of the test set up (specimens with normal compressive stress), (c) Position of LVDTs to measure shear slip and crack opening.
3.3 General Observations

Tests have shown that the design of specimens was conservative enough to avoid any parasitic or premature cracking in locations other than along the interface or its vicinity. In all specimens a crack opened along the interface between the two concrete blocks and it was visible even for an imposed shear slip as small as 0.10mm-0.20mm. The observed failure modes are shown in Figure 3. The specimens made of steel fiber reinforced concrete have reached higher values of the interface resistance; the maximum resistance was mobilized at larger values of the shear slip than the specimens made of plain lightweight concrete; it was possible to apply larger values of the shear slip, without failure of the interface. On the other hand, in the specimen subjected to compressive stress normal to the interface, the crack opening remained small. That specimen reached significantly higher shear resistance than the identical specimen not subjected to normal stress (Table 1).

In almost all specimens, diagonal cracks opened in the “new” part of the specimen, at imposed shear slip values smaller than 0.50mm. In most cases, in the small concrete block (cast approximately one month after the first one) the cover of the reinforcement was significantly smaller. The cracks initiated from the interface (at the position of the one or more bars) and propagated at an angle of approximately 45º within the small concrete block, up to the block edge. The opening of the diagonal cracks was rather limited in case of specimens made of steel fibre reinforced concrete. In the specimen reinforced with rebars having an embedment length equal to 12.5d, a crack parallel to the interface, close to the end of the embedment length, led to failure.

Figure 3: Observed Failure Modes: Diagonal cracks initiated from the interface and propagated at an angle of approximately 45º. Formation of a second crack, parallel to the interface.

3.4 Hysteresis loops and maximum shear resistance

Figure 4 shows the hysteresis loops for the tested interfaces. All features that are typical for shear sensitive elements may be observed: Pronounced pinching effect associated with limited area of hysteresis loops (limited energy dissipation) and substantial force response degradation due to cycling. Those characteristics become more pronounced for limited embedment length of the rebars, for smaller bar diameter, for concrete without fibres and for specimens not subjected to compressive stress. For small values of the applied shear slip, the shear response degradation is rather limited whereas the behavior of the interface is practically linear.
Another feature, typical of specimens with insufficiently anchored bars, is the pronounced asymmetry of the hysteretic loops in the two loading directions. Actually, as shown in Figure 4 (specimen R-34L/E/20/0.1), the resistance mobilized in the second loading direction may be as low as half the resistance mobilized in the first loading direction. On the contrary, in some cases, the resistance mobilized in the second loading direction may be higher than the resistance mobilized in the first loading direction. This higher resistance was not recorded since the beginning of testing, it was observed when a diagonal crack was formed. Therefore, the unexpected behaviour can be attributed to the existence of the crack, which led to larger measured shear slip values than those actually applied on the first direction of loading.

As shown in Figure 4, the behaviour of lightweight concrete specimens is comparable with the behaviour of normalweight concrete specimens. It is noted that the behaviour of specimens made of concrete without steel fibres can be considered inferior, given that the maximum interface resistance of these specimens was not significantly smaller, but the maximum value of the applied shear slip was limited. The failure of these specimens was more brittle, and was due to the formation of inclined cracks. In addition, in case of lightweight concrete specimens, the pinching effect was more pronounced and the energy dissipation was smaller.

The maximum interface resistance is presented in Table 1: As expected, the maximum mobilized shear resistance increases with increasing embedment length of the bars. The positive effect of the normal compressive stress on the interface and of the addition of steel fibres may also be observed.

In Table 2, the maximum mobilized shear resistance per cycle is given, for LRC interfaces reinforced with 12mm rebars and the relevant RC interfaces. The force response at each cycle is taken as the
average response in the two loading directions. In Table 3, the force response at the n-th cycle, $V_n$, normalized to that of the first cycle, $V_1$, is given as a function of the number of cycles n. The force response, as well as its degradation due to cycling, is given for the applied shear slip values. The value of the ratio $V_n/V_1$ depends, as expected, on the concrete type, on the embedment length of bars, on the normal compressive stress on the interface as well as on the addition of steel fibres on the concrete. The positive effect of the presence of compressive normal stress on the interface is obvious, given that the degradation of the response is negligible, even for large values of the applied shear slip. The effect of cycling on the mobilized shear resistance is also illustrated in Figure 5, where the hysteresis loops envelopes are shown for the first and the second loading cycles.

It is noted that the behaviour of LRC specimens constructed with steel fibres differs significantly from the behaviour of those without: a) The maximum value of the applied shear slip is larger for these specimens; and b) Their failure mode is differs in that cracking is concentrated along the interfaces, while diagonal cracking does not lead to the termination of the tests.

Table 2 Maximum shear resistance mobilized per cycle (average response of the two directions).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>s=±0.10mm</th>
<th>s=±0.20mm</th>
<th>s=±0.40mm</th>
<th>s=±0.60mm</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>n=1 2 3</td>
<td>n=1 2 3</td>
<td>n=1 2 3</td>
<td>n=1 2 3</td>
</tr>
<tr>
<td>R-34L/E/12/0.1</td>
<td>81.2 88.2 75.8</td>
<td>103.2 94.8 95.1</td>
<td>135.0 122.3 115.1</td>
<td></td>
</tr>
<tr>
<td>R-34L/E/20/0.1</td>
<td>83.8 82.3 80.6</td>
<td>114.8 117.7 114.4</td>
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<td></td>
</tr>
<tr>
<td>NR-34L/E/20/0.1</td>
<td>70.4 73.1 69.3</td>
<td>110.2 100.7 100.7</td>
<td>157.6 125.6 131.3</td>
<td>137.1 135.2 135.1</td>
</tr>
<tr>
<td>R-34L/E/20/0.1</td>
<td>98.4 121.8 93.8</td>
<td>160.3 148.7 148.0</td>
<td>237.9 225.7 215.7</td>
<td>261.9 237.8 228.0</td>
</tr>
<tr>
<td>R-31/E/20/0.1</td>
<td>99.1 78.2 75.6</td>
<td>128.1 116.7 113.7</td>
<td>173.2 142.6 132.2</td>
<td>181.1 151.0 141.4</td>
</tr>
<tr>
<td>NR-25/E/20/0.1</td>
<td>98.9 153.7 167.9</td>
<td>168.5 215.7 166.5</td>
<td>195.2 159.9 120.0</td>
<td>132.1 189.3 155.2</td>
</tr>
<tr>
<td>R-30f/E/20/0.1</td>
<td>111.1 104.8 100.6</td>
<td>151.0 134.8 128.2</td>
<td>189.8 167.5 152.1</td>
<td>196.9 169.2 161.6</td>
</tr>
</tbody>
</table>

Table 3 Force response degradation due to cycling; $V_n/V_1$ values

<table>
<thead>
<tr>
<th>Specimen</th>
<th>s=±0.10mm</th>
<th>s=±0.20mm</th>
<th>s=±0.40mm</th>
<th>s=±0.60mm</th>
</tr>
</thead>
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<tr>
<td></td>
<td>n=1 2 3</td>
<td>n=1 2 3</td>
<td>n=1 2 3</td>
<td>n=1 2 3</td>
</tr>
<tr>
<td>R-34L/E/12/0.1</td>
<td>1.00 1.09 0.93</td>
<td>1.00 0.92 0.92</td>
<td>1.00 0.91 0.85</td>
<td>1.00 0.91 0.85</td>
</tr>
<tr>
<td>R-34L/E/20/0.1</td>
<td>1.00 0.98 0.96</td>
<td>1.00 1.03 1.00</td>
<td>1.00 0.91 0.91</td>
<td>1.00 0.80 0.83</td>
</tr>
<tr>
<td>NR-34L/E/20/0.1</td>
<td>1.00 1.04 0.99</td>
<td>1.00 0.91 0.91</td>
<td>1.00 0.80 0.83</td>
<td>1.00 0.99 0.99</td>
</tr>
<tr>
<td>R-34L/E/20/0.1</td>
<td>1.00 1.24 0.95</td>
<td>1.00 0.93 0.92</td>
<td>1.00 0.95 0.91</td>
<td>1.00 0.91 0.87</td>
</tr>
<tr>
<td>R-31/E/20/0.1</td>
<td>1.00 0.79 0.76</td>
<td>1.00 0.91 0.89</td>
<td>1.00 0.82 0.76</td>
<td>1.00 0.83 0.78</td>
</tr>
<tr>
<td>NR-25/E/20/0.1</td>
<td>1.00 1.55 1.70</td>
<td>1.00 1.28 0.99</td>
<td>1.00 0.82 0.61</td>
<td>1.00 1.43 1.17</td>
</tr>
<tr>
<td>R-30f/E/20/0.1</td>
<td>1.00 0.94 0.91</td>
<td>1.00 0.89 0.85</td>
<td>1.00 0.88 0.80</td>
<td>1.00 0.86 0.82</td>
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### Table

<table>
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<tr>
<th>Specimen</th>
<th>$s=\pm0.80\text{mm}$</th>
<th>$s=\pm1.00\text{mm}$</th>
<th>$s=\pm1.20\text{mm}$</th>
<th>$s=\pm1.40\text{mm}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>n=1</td>
<td>2</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>R-34L/E/20/0.1</td>
<td>1.00 0.95 0.90</td>
<td>1.00 0.90 0.83</td>
<td>1.00 0.90 0.81</td>
<td>1.00 0.90 0.81</td>
</tr>
<tr>
<td>NR-34L/E/20/0.1</td>
<td>1.00 0.83 0.78</td>
<td>1.00 0.87 0.85</td>
<td>1.00 0.87 0.85</td>
<td>1.00 0.75 0.66</td>
</tr>
<tr>
<td>R-31/E/20/0.1</td>
<td>1.00 0.85 0.78</td>
<td>1.00 0.81 0.80</td>
<td>1.00 0.81 0.80</td>
<td>1.00 0.68 0.56</td>
</tr>
<tr>
<td>NR-25/E/20/0.1</td>
<td>1.00 0.85 0.82</td>
<td>1.00 0.71 0.70</td>
<td>1.00 0.71 0.69</td>
<td>1.00 0.71 0.69</td>
</tr>
</tbody>
</table>

**Figure 5:** Hysteresis loop envelopes for specimens constructed with lightweight concrete compared with specimens constructed with normal concrete: (a) first cycle, (b) second cycle.

### 3.5 Calculation of the maximum shear resistance

In the Literature, as well as in regulatory documents, several relationships are given for the prediction of the maximum interface resistance. The proposed relationships are validated for interfaces in normalweight concrete. Up to now, the validity of those relationships was not checked for interfaces within lightweight concrete elements. Among the numerous relationships available in the Literature, a relationship proposed by Tassios and Vassilopoulou, 2003\(^{18}\), was selected by Palieraki et al., 2014\(^{16}\) for further investigation. Taking into account the database, the authors\(^{16}\) have proposed a set of modified factors to be implemented in Equ. (1):

$$\tau_u = \beta_d \tau_d + \beta_f \tau_f$$  \(\text{(1)}\)

Where, $\beta_d$ and $\beta_f$ are contribution factors for the mechanisms acting along the interface, namely friction and dowel action. On the basis of the re-evaluation of the experimental results from the Literature\(^{16}\), a set of contribution factors are proposed, taking into account the specific characteristics of various types of interfaces. Further modifications are proposed for the contribution factor $\beta_f$ for the contribution of the friction mechanism (Table 4), to account for the different behaviour of lightweight concrete interfaces. The contribution factor $\beta_u$ for the dowel action, is always taken equal to 0.70, for rebars with embedment length larger than 6d. It is noted that the interfaces within lightweight concrete containing steel fibres exhibit a behavior similar to normalweight concrete.
interfaces. The comparison between experimental and predicted values of the maximum resistance of lightweight concrete interfaces is shown in Figure 6. It seems that the experimental values are quite accurately predicted.

Table 4 Interfaces between old and new concrete: Contribution factors for the friction mechanism.

<table>
<thead>
<tr>
<th>Interface Characteristics</th>
<th>$\beta_f$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rough Interface, normalweight concrete</td>
<td>0.60</td>
</tr>
<tr>
<td>Smooth interface, normalweight concrete</td>
<td>0.40</td>
</tr>
<tr>
<td>Rough Interface, lightweight concrete</td>
<td>0.40</td>
</tr>
<tr>
<td>Smooth interface, lightweight concrete</td>
<td>0.20</td>
</tr>
<tr>
<td>Rough interface, cyclic loading</td>
<td>0.40</td>
</tr>
<tr>
<td>Smooth interface, cyclic loading</td>
<td>0.20</td>
</tr>
</tbody>
</table>

![Figure 6: Comparison between experimental shear resistance and values calculated on the basis of the proposed modified formula (lightweight concrete interfaces).](image)

4 Conclusions

The experimental results presented in this paper allow for the following conclusions to be drawn:

(1) Cyclically slips imposed to lightweight concrete interfaces lead to significant force-response degradation. The amount of response degradation is a function of the diameter of the reinforcement, the anchorage length of the reinforcing bars, the presence of stress normal to the interface and the addition of steel fibres in the concrete.

(2) The tests confirm the positive effect of the presence of compressive stress normal to the interface. A compressive stress acting on the interface leads to increased maximum shear resistance, as well as to reduced degradation due to cycling. Steel fibres added to lightweight concrete have a similar effect on the behavior of the interface.

(3) The behavior of lightweight concrete specimens is comparable with the behavior of normalweight concrete specimens. Finally,
(4) The calculation of the shear resistance using the formula proposed by the first author seems to be quite accurate.

5 Acknowledgement

The current paper includes part of the results of the Research Project 716-BET-2013 “Structural Lightweight High Performance Concrete with Pumice Aggregates”. This project is being supervised by the General Secretariat for Research and Technology and it is partially funded by the Greek State and the EU under the action NSFR 2007-2013 “PABET 2013”. The findings and the conclusions are the sole responsibility of the authors.

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DEVELOPMENT AND APPLICATION OF A HEAVY DUTY ANCHOR SYSTEM

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ABSTRACT

Safe anchoring of components with heavy loads, strong earthquakes or explosion impacts represents a great challenge to engineers. For small to medium loads (< 6t) a wide range of approved products such as headed studs or anchors are available, and can be conveniently applied. However, for post-installed anchorages with heavy loads during retrofitting or strengthening of existing concrete components, the HOCHTIEF Heavy Duty Anchor System (HT-SHV) is a reliable and fast to install solution for allowable characteristic tensile forces up to 175 kN (M30 version) respectively 600 kN (M48 version).

This paper deals with the development of the HT-SHV system with discrete end anchorage for extraordinary wide cracks up to w = 1.5 respectively 3 mm. It also highlights the adaption for widespread applications: As the system permits adjustable embedment depth from 20 cm to 2.8 m (M30 version) respectively 75 cm to 5 m (M48 version), the load transfer within or across structural components can be tuned in multiple ways. The derivation of these solutions will be shown.

1 Introduction

In heavy industry and power plants, especially nuclear power plants (NPPs), safe anchoring of components, for example those under heavy loads such as strong earthquakes or explosions impacts, represents a great challenge to engineers. For small to medium loads (< 6 t) a wide range of approved products such as headed studs or anchors are available. However, for post-installed anchors with heavy loads during retrofitting or strengthening of existing concrete members or fastening of heavy structural and non-structural components, the HOCHTIEF Heavy Duty Undercut Anchor System (HT-SHV) shown in Figure 1 is a reliable and fast-to-install solution.

The anchor system is available in two versions: HT-SHV/30c and HT-SHV/50s with different load-bearing capacities. Both consist of a steel rod with end anchorage placed in a diamond cored undercut drill hole which is filled with a special grout. The other end is fixed with a hexagon nut, a washer and a counter-nut. The anchor is used to subsequently introduce very high loads in concrete construction components or retrofitting of concrete constructions. Figure 2 shows the principle of the undercut end anchoring on the example of the HT-SHV/30c.
Figure 1: Versions of the HT-SHV system

Figure 2: Visualisation of undercut anchorage using HT-SHV/30c as an example (HT-SHV/50s analogically)
2 Special features

The HOCHTIEF Heavy Duty Undercut Anchor provides a form-fitting end anchorage system using high-strength mortar in a specifically formed undercut. Therefore, applying a large prestressing force is possible. It is also capable of carrying shear loads. The load-bearing capacity design is based on the European Guideline ETAG 001-Appendix C\(^1\). The HT-SHV/30c system has been approved by the German building authority DIBt under No. Z-21.8-1920\(^3\). The application for NPPs under operational and exceptional loads like earthquake loads according to the German Guideline for fastenings in NPPs\(^2\) is authorised by approval in individual cases via expert appraisals and preceding approval tests. The HT-SHV/50s also requires approvals in individual cases which have been granted before due to expert appraisals and approval test as well. Table 1 shows the major data of the two available versions.

Table 1: Versions of HOCHTIEF Heavy Duty Undercut Anchor system

<table>
<thead>
<tr>
<th></th>
<th>HT-SHV/30c</th>
<th>HT-SHV/50s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geometric values</td>
<td>thread M30, core drilling Ø 67 mm</td>
<td>thread M48, core drilling Ø 120 mm</td>
</tr>
<tr>
<td>End anchorage</td>
<td>cold-formed foot like headed stud</td>
<td>load distributing anchorage plate at the end acting like headed stud</td>
</tr>
<tr>
<td>Allowable characteristic tension force</td>
<td>N(_{\text{Ek}}) = 175 kN</td>
<td>N(_{\text{Ek}}) = 600 kN</td>
</tr>
<tr>
<td>Allowable crack width(^1)</td>
<td>w \leq 1,0 mm / 1,5 mm(^2)</td>
<td>w \leq 3,0 mm</td>
</tr>
<tr>
<td>Minimum required(^3) embedment depth h(_{\text{ef}})</td>
<td>20 cm</td>
<td>75 cm</td>
</tr>
<tr>
<td>Completed maximum embedment depth h(_{\text{ef}})</td>
<td>2,8 m</td>
<td>5,0 m</td>
</tr>
</tbody>
</table>

1) For operational and exceptional loads
2) w = 1,0 mm during service condition, w = 1,5 mm during suitability tests according to Guideline\(^2\)
3) Reduction of allowable tension force may be necessary depending on concrete grade

3 Load-bearing behaviour

Tension loads are transferred into concrete members with a combination of different bearing mechanisms as shown in Figure 3. There are shear forces between the mortar and the existing concrete as well as multiaxial states of stress in the regions of the end anchorage and the undercut. The ultimate failure mode depends mainly on the embedment depth. Shear loads can also be transferred to the concrete structure.
Different tests of the load-bearing behaviour of the HT-SHV/50s have been carried out during the approval procedure at MPA (Public material research laboratory) Stuttgart, Germany; failure loads of approximately 1000 kN were reached. The anchorage system failed because of concrete cone cracking as shown in Figure 4(a). No pull-out of the steel rod occurred and only small deformations were measured before reaching the ultimate load. The results match the ones received from an FE analysis carried out during the approval process, shown in Figure 4(b).

Figure 3: Load-bearing behaviour using HT-SHV/30c as an example
(continuous lines: due to tension load, dashed lines: due to shear load)

Figure 4: (a) Approval test of HT-SHV/30c at MPA (Public material research laboratory) Stuttgart, Germany, embedment depth $h_{ef} = 500$ mm, uncracked concrete, typical failure cone $\varnothing 2$ m,
(b) FE analysis of HT-SHV/50s, Tension strain in z-direction

Figure 5 and Figure 6 show the resulting deformations during the approval tests of both versions of the HOCHTIEF Heavy Duty Undercut Anchor.
Before installation, the embedment depth must be marked on the anchor rod. The embedment depth of the system is variable; for the HT-SHV/30c anchor if applied according to the approval between $h_{ef} = 200$ mm and 600 mm. Deeper embedment is possible for both systems. The drill hole for the anchor is diamond cored and produced in 2 steps. In the first step a cylindrical drill hole of diameter $d_0 = 67$ mm is cored. The cylindrical drill hole must be deeper than the embedment depth of the anchor used in design. Damage of reinforcement can be tolerated. In the second step the undercut is
made with a special undercutting tool (Figure 7(b)). The bits expand and cut through the surrounding material while the tool is rotating until the final shape of the undercut is completed. The maximum diameter of the undercut is then 103 mm. Both undercutting tool and drill rig can be adjusted to the chosen embedment depth. After drilling and undercutting it can be controlled with a special gauge or a borescope if the undercutting was performed successfully (Figure 7(c)). The drill hole is cleaned by rinsing with water. The surface quality is inspected with an optical square or with a borescope. If everything is done properly the anchor can be set, fixed and cast in using special mortar. The mortar is a high strength, low-shrink, cement based mortar with a high early strength. Loading or potential prestressing of the anchor is possible 24 hours after casting using reduced loads. Within 7 days after casting full load application is possible.

For horizontal and overhead installation special devices are available to fix the anchor rod during hardening and to guarantee the complete casting of the drilled hole and the anchor rod (Figure 8 and Figure 9). Special vents at the end of the anchor and vent hoses ensures no air remains in the drilled hole.
5 Example of use: Crane anchoring NPP Gösgen (Switzerland)

The bracket crane in the annular space of the reactor building had to be strengthened for higher loads and mounted for earthquake loads in its parking position. There were two possible solutions for this. Either the crane could have been mounted by plugging anchor rods through the whole concrete section and securing it with lock nuts or by using the HOCHTIEF Heavy Duty Undercut Anchors to mount the crane without penetrating the concrete section completely. The latter method was chosen. The resulting anchoring at the two connection points on the dome was designed and verified. Therefore, a catch hook was welded to an anchor plate and anchored with 12 HOCHTIEF Heavy Duty Undercut Anchors and 12 shear cams made of steel pipes of length 70 mm. The safety against earthquake provided by this prohibits unexpected movement of the crane during exceptional load cases. The crane is locked in its parking position by the lower carriage at the catch hooks, which transfer loads acting vertically and transverse to the rail. See Figure 10 for more details.

The dimension of the anchor plate is 1500 mm x 800 mm x 50 mm. The shear cams are welded to it on the grouting side before the anchors can be inserted and grouted (Figure 11(b)). Several vent
holes are arranged for this process. Due to the chosen connection, it is possible to bear shear forces completely between the anchor plate and the shear cams. Tension forces are carried by the anchors. A tolerance zone for the placement of the anchors has been determined for the primary verifications because the existing reinforcement must not be damaged. The exact placement could only be determined after a scan of the reinforcement with radar and exploration openings.

A mortar layer of 13 mm to 23 mm is applied underneath the anchor plate. The exact value depends on the radius of the reactor dome and the planned tolerance for the assembly of the concrete surface. HOCHTIEF Heavy Duty Undercut Anchors HT-SHV/30c with an effective embedment depth \( h_{ef} = 600 \) mm are used.

Earthquake loads due to the dead load of anchor plate and crane have to be transformed into the plane of the plate. The verifications were carried out in two steps. The first one was an idealised calculation of the anchors and the plate for the estimated worst case situation. The verification was carried out with those six anchors which were closest to the point of load application because they have to carry the highest loads. Figure 12 shows the position of load introduction and the anchors. Stresses in the anchor plate as well as tension and shear forces of the anchors and shear cams were calculated during a non-linear FE-analysis for different load positions.
Furthermore, verifications considering horizontal sliding of the crane along the rails of a magnitude of 500 mm were carried out. The four anchors being closest to the point of load application were considered for this. The shear cams were verified against the resulting shear forces from the load and torsion due to the eccentric load introduction. The second step of the calculation was an As-Built evaluation of the anchor plates. In doing so, the exact position of the anchors and shear cams according to recordings of the bore holes were used for the verifications.

6 Strengthening of foundations and heavy load anchorage

The HOCHTIEF Heavy Duty Undercut Anchor can be used in heavy industry, for example refitting of steel mills: Fastenings of new machinery or strengthening of foundations are performed examples. Large and potentially dynamic forces have to be transmitted into the foundations due to the heavy machines. This can be achieved by HOCHTIEF Heavy Duty Undercut Anchors. Figure 13 to Figure 15 show another example of how to use long HT-SHV anchors (up to 5m length) for strengthening of foundations as well as fastening of machinery.

In Figure 13 the foreseen load transfer can be seen of high accidental loads ($F = 3000$ kN) from the main drive in horizontal direction to the base plate. For the vertical tension forces of up to 1200 kN in each corner two HT-SHV anchors per edge were used. The anchors have their end anchorage in the base plate with embedment depth in the base plate of less than 1 m. The rest of the anchorage length of up to 4 m was carried out without bond to the grout to ensure the end anchorage in the base plate and to allow a pretension of the anchors ($P = 600$ kN). For the horizontal loads external tendons were used.

![Figure 13: Retrofitting of steel mills, strengthening of drive foundation for exceptional loads, (a) load transfer, (b) Anchorage length $h_{ef} > 4$ m](image_url)
Figure 14 shows the fastening of the main drive on the same foundation for accidental torsional loads (Tension load $F = 1050$ kN). On one side of the foundation, openings to the center of the foundation were present. The openings were closed force-fit to transfer compression, but they limited the end anchorage (less than 1 m) to the level of the base plate only. Again the rest of the anchorage length of up to 4 m was carried out without bond to the grout to ensure the end anchorage in the base plate and to allow a pretension of the anchors ($P = 600$ kN). Figure 15 shows the same application for another type of foundation and machinery.

Figure 14: Retrofitting of steel mills, fastening of main drive for exceptional loads

Figure 15: Retrofitting of steel mills, (a) Anchorage of refitted hot plate leveler, (b) strengthening of foundation for exceptional loads
Another field of application is the restructuring of steel wind turbine anchorage systems. By adding a new layer of concrete on top of the existing foundation it is possible to brace the two parts subsequently to achieve a better transmission of forces into the foundation by prestressing. The connection and prestressing is achieved by using HOCHTIEF Heavy Duty Undercut Anchors. This is shown in Figure 16.

![Figure 16: Restructuring of the anchorage of steel wind turbines with a foundation mounting part](image)

7 **Further possible fields of application**

The HOCHTIEF Heavy Duty Undercut Anchor can be used for dismantling huge concrete blocks from constructions such as radiation shields of NPPs. Single anchors were used to carry blocks with a weight up to 30 t. The drill holes are prepared while the shield is still intact and after applying the anchors the concrete blocks are cut off and lifted as shown in Figure 17.

![Figure 17: (a) Dismantling of NPPs (b) Lifting of concrete radiation shield parts](image)
8 Conclusion

The HOCHTIEF Heavy Duty Undercut Anchor is an efficient tool to introduce high loads into cracked and uncracked concrete components. Two different sizes of the anchor are available: the smaller HT-SHV/30c with a maximum allowable characteristic tension force $N_{Ek} = 175$ kN and the larger HT-SHV/50s with $N_{Ek} = 600$ kN. The former is approved by the German building authority DIBt (German Institute for Civil Engineering) in structural engineering projects. Load bearing tests as well as numerical analyses have been carried out for both versions of the anchor. They have been applied at NPPs under operational and exceptional loads like earthquakes in Germany and foreign countries many times after authorisation by expert appraisals, preceding approval tests and approval in individual cases.

Both versions of the anchor can be applied in many different situations and provide efficient solutions to introducing high loads into concrete structures.

References:


DEVELOPMENTS FOR DESIGN
PERFORMANCE BASED APPROACH FOR ANCHORAGE IN CONCRETE CONSTRUCTION

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ABSTRACT

The design of anchorages in concrete is traditionally done using force based method, where the design load for an anchorage is calculated corresponding to different failure modes and the least of these values defines the allowable load that can be applied for the anchorages. This approach does not allow or account for any redistribution of the forces in the anchorage due to possibly ductile behavior that can be achieved in case of anchor steel failure or reinforcement failure. This might lead to a non-uniform level of safety in different configurations of anchorages. Furthermore, specifically with respect to the anchorages used to connect the elements required for strengthening (esp. seismic strengthening) of existing structures, the force based design methods often lead to no solutions or unreliable solutions.

This paper introduces a performance based approach that can be used for the design or assessment of the anchorages. The approach consists of considering not only the load-carrying capacity of the anchorage but also the displacement behavior of the anchors so that a redistribution of the forces can also be accounted for. Different methods such as component model or an equivalent spring model that can be used for the assessment are discussed. It is demonstrated with the help of examples how this approach can be used for a reliable and efficient design/assessment of the anchorages. It is emphasized through an example that the displacement behavior of the anchors has a significant influence on the performance of the anchorage used in seismic strengthening even though the load carrying capacity of the anchorage is the same. This shows that the force based design methods for the anchorages might not only be inadequate but also unreliable in certain cases.

1 Introduction

Anchorages or fastenings are often used in concrete construction to connect the structural or non-structural components to the main structure. The non-structural components often connected using the anchorages include machinery, piping, tanks or silos, facades, false ceilings etc. Examples of the structural components connected using anchors include structural members e.g. steel beams or columns connected to concrete walls or foundations respectively, components for strengthening e.g. steel bracing, steel haunch elements, steel jackets, etc. In order to ensure a reliable transfer of forces from the main structure to the component and vice-versa, it is essential that the anchorage connecting the two systems serve its purpose well. If the anchorage is unable to make the transfer of forces in a reliable manner, the whole system might render useless or even counter-productive.
Traditionally, the anchorages are designed following a force (or resistance) based design approach in which the failure loads corresponding to different possible failure modes for the anchorage are calculated and the smallest failure load governs the design load\(^1\). While calculating the failure loads, it is assumed that all the anchors in an anchor group have equal stiffness and they are all loaded through an anchor plate which is stiff\(^2,3\). When an anchor group in cracked concrete is considered, it is assumed that all the anchors in the group are intercepted by a crack and all the anchors in cracked concrete also have equal stiffness. Under concentric loading, these assumptions lead to a uniform force distribution among all the anchors. Under eccentric loading, where the force distribution among the anchors is non-uniform, the highest loaded anchor is verified and the failure load associated with its failure defines the design failure load for the entire group.

The force based design approach means that once an anchor fails no re-distribution of the forces among the anchorages is considered. This may lead to over-conservative results and in certain cases no design solution can be arrived at e.g. in case of strengthening components connected with the anchorages. In reality, there is always a possibility of re-distribution of the forces among the anchors in a group once an anchor loses stiffness due to overloading. However, the extent of redistribution depends on the load-deformation behavior of the anchors, which in turn depend on the failure mode, deformation behavior of the anchorage material (concrete), presence or absence of reinforcement etc. In addition, a major factor deciding the extent of redistribution is the base plate thickness itself.

Currently, no explicit guidelines are available stating the requirements of the thickness of base plate to be considered as stiff other than the qualitative statements that the base plate shall remain linear elastic and that the deformations of the base plate shall be negligible compared to the axial deformations of the anchors\(^2\). This means that the same base plate which could be considered stiff for an anchor group configuration with a particular type of anchors, which are flexible or undergo larger deformations, might not be stiff enough for the same anchor group configuration with other type of anchors that are relatively stiff.

Furthermore, currently for an anchorage either of the two conditions are considered: (i) None of the anchors in intercepted by a crack (uncracked concrete), and (ii) All the anchors intercepted by the crack (cracked concrete). In reality, it is possible that only some of the anchors are intercepted by the crack and in certain cases, this could lead to lower failure loads than while considering all the anchors in the crack due to the difference in stiffness of the anchors in uncracked and cracked concrete. A strength based approach, in principle, cannot consider the influence of only some and not all anchors intercepted by the crack. Currently EN1992-4\(^2\) gives an indirect consideration to this effect only under seismic loads through a reduction factor \(\alpha_{eq}\) applied for the anchor groups.

The above-stated examples highlight that even though a strength based design approach for the anchors is a good and easy design tool, it has several limitations due to which the uniform safety margins are very difficult to achieve.

This paper presents a concept for a possible performance based approach that can be used for the design or assessment of the anchorages. The approach consists of considering the not only the load-carrying capacity of the anchorage but also the displacement behavior of the anchors. The concept is extendable to potentially eliminate the above-stated limitations of force based approach.
2 The concept of performance

2.1 Structural performance

In structural engineering, the performance based approach is most developed for the case of structures subjected to seismic loads. The performance of a structure subjected to an earthquake is judged by the extent of damage incurred in the structure due to the earthquake. Principally, the performance of a structure is related to the displacement of the structure. A typical plot of base shear v/s a reference displacement (e.g. roof displacement) is shown in Figure 1.

In the beginning, the applied base shear rises linearly and steeply with the reference displacement. In this linear elastic range, the structure suffers almost no damage and if the structure is unloaded at this stage, theoretically, it should be able to attain its original undeformed shape without any residual deformations. This performance level is referred to as “Operational” performance level and signifies that the structure is fully operational and in almost same condition after the earthquake as it was prior to the earthquake. If the applied load on the structure is increased, the reference displacement will start displaying a nonlinear relation with the base shear. At this level, significant cracking can be obtained at certain locations e.g. sections where high bending moments arise due to the applied lateral loads. If the structure is unloaded at this point, permanent deformations will remain and the damage (cracking) may not be reversible. Such damages are generally not very severe and the structure may be easily repairable. Such a performance level characterized by minor repairable and/or non-structural damage in the structure is referred to as “Immediate Occupancy” performance level which signifies that the performance of the structure is such that it can be immediately occupied again after the earthquake. In a typical reinforced concrete frame structure, at this stage, flexural cracks at the ends of the beams and columns of lower storeys can be expected.

The behavior of the structure beyond this point is characterized by a rapid increase in reference displacement with small increments of applied load. Physically, the structure will be subjected to
significant damage with gradual propagation and opening of existing cracks as well as formation of new cracks. The damage is irreversible and the structure may or may not be repairable. This performance level is often referred to as “Life Safety” performance level, which means that at this stage, the probability of complete or partial collapse of structure is low and therefore the safety of the persons living inside is not endangered. On further pushing the structure, often, a gradual drop in the load with corresponding increase in roof displacement can be observed. In this range, large damage can be observed at various potential plastic hinge locations in the structure. Such a performance level is referred to as “Collapse Prevention” performance level, which implies that the structure is pushed to verge of incipient collapse but actual collapse is prevented. Any further lateral push would result in the collapse of structure.

Although performance is a qualitative concept, as seen above, it is strongly related to the displacements, which is a quantifiable parameter. Therefore, for the estimation of performance, the analysis has to be performed in the displacement domain and through the complete range of displacements, linear and nonlinear.

2.2 Extension to anchorage performance

The general concept of performance can be relatively easily extended to the anchorages as well. In order to do so, similar to the case of structures, analysis must be performed in displacement domain. Traditionally, the anchorage design is performed in force domain by calculating the failure load corresponding to different failure modes and considering the least of them in design. However, different types of anchors have different stiffness values and therefore displacement behavior, e.g. bonded anchors and concrete screws typically display significantly high stiffness compared to the expansion anchors. These aspects are indirectly accounted for through qualification tests.

Unlike the current force based approaches, the performance based design of the anchorage requires the information of the complete load-displacement behavior of the anchors. The load-displacement behavior of an anchor depends strongly on the failure mode, e.g. steel failure displays a ductile load-displacement curve and concrete cone breakout is associated with a quasi-brittle load-displacement response. Figure 2 explains the concept of the dependence of the performance of an anchorage on its load-displacement behavior.

Consider an anchorage with three anchors in a row loaded in centric and eccentric tension. Two cases are considered here, namely, the anchorage with a stiff base plate and the anchorage with a flexible base plate. The actual failure load that can be achieved by the anchorage in such a case will depend on the load-displacement behavior of the single anchors. In Figure 2, it is assumed that the spacing between the anchors is equal to the critical spacing. Two extreme possibilities of the load-displacement behavior of the individual anchors are considered, namely, perfectly elasto-brittle and perfectly elasto-plastic. These extreme scenarios are only theoretical and used here for explanation purposes only. The real fracture is always quasi-brittle in nature. The stiffness of the anchors is considered equal for all the anchors. For simplicity, the displacement profile of the base plate is also idealized. It is clear from Figure 2 that the load-displacement behavior of an anchor, especially the descending branch has a significant influence on the performance of the entire anchorage. The reason for this influence is the possibility of redistribution of forces to the other anchors after the failure of the first anchor in case of ductile fracture.
Figure 2: Theoretical explanation of performance based approach on anchorage with three anchors in a row with (a) stiff base plate and centric load, (b) stiff base plate and eccentric load, (c) flexible base plate and centric load, and (d) flexible base plate and eccentric load.
To verify these theoretical considerations, experimental investigations were performed on an anchorage with three anchors in a row in normal concrete and steel fiber reinforced concrete (SFRC). SFRC is known to have significantly higher fracture energy and a comparatively ductile load-displacement behavior in tension compared to normal concrete. The complete study is presented in Bokor et al. The tests were performed on anchorage with stiff and flexible base plate and under centric and eccentric loading. However, in the tests, the spacing between the anchors was less than the critical spacing. It was shown in these tests that even though the load carrying capacity of the single anchors in SFRC was at an average 18% higher than the capacity of the single anchors in normal concrete, the capacity of the anchor groups in SFRC was more that 40% higher than the capacity of the corresponding group in normal concrete. This increase in the capacity is attributed to the more ductile fracture of SFRC compared to normal concrete.

Additionally, it was found that due to the reduction of the anchor plate thickness from 50mm to 25mm, the peak loads decreased by 12% in normal concrete and only 1% in SFRC. After the (most loaded) outer anchor of the group failed in normal concrete, the peak load could not be increased further, because the capacity of the other neighboring anchors only influenced the post-peak behavior of the group and they did not contribute to the ultimate load. On the contrary, in the case of anchor groups in SFRC, an increment of the ultimate group capacity was observed after the first cracking. More details about the tests can be found in reference 4.

3 Possible approaches for performance analysis of anchorages

As mentioned earlier, for the performance analysis of the anchorages, it is essential to obtain the complete load-displacement characteristics of the anchors and not just the peak loads. Although there are not many studies performed on these lines, there are certain approaches which are available and can be used for carrying out the displacement based analysis of the anchors. Two of such approaches are discussed below. Although not every application is discussed here, the approach in general, is extendable to eliminate most of the weaknesses of the force based approach.

3.1 Component model

Component model basically segregates the load-deflection behavior of the anchor into the behavior of different components, namely (a) steel (Component S), (b) pullout (Component P), (c) Concrete (Component S), and (d) Supplementary reinforcement (Component RS/RB), if applicable. The concept of the component model for anchorages in concrete was primarily developed within the framework of the Infaso and Infaso+ projects 5,6 and is depicted in Figure 3.

In the component model, the different components are modeled through springs that are assigned non-linear load-displacement characteristics. The steel component (Component S) considers the elongation of the shaft of the anchor and is assigned an elastic-perfectly plastic load-displacement behavior with the ultimate load, N_ys, corresponding to the yield capacity of the anchor shaft. The steel component is connected to the nonlinear spring representing the pullout component (Component P) of the anchor. The pullout component has characteristics that are nonlinear from the beginning with a gradually reducing stiffness until the peak load corresponding to pullout, N_up, is reached. The springs for steel and pullout components are further connected in series with the springs for the concrete component (Component C), which is in turn connected in parallel with the spring for
the supplementary reinforcement (Component RS/RB), if applicable. The concrete component is idealized through a spring with an infinite stiffness until the concrete cone capacity, \( N_{uc} \), is reached followed by a linear descending branch.

![Schematic of the component model for anchorages in concrete developed within the framework of Infaso and Infaso+ projects](image)

In case of the presence of supplementary reinforcement, the concrete component is connected in parallel with the component of reinforcement, which consists of either the component of steel failure of the supplementary reinforcement (component RS) or bond failure of the supplementary reinforcement (component RB). The spring characteristics are idealized through a nonlinear ascending branch until the failure load, \( N_{u_re} \), followed by a constant horizontal line.

Further details of the component model can be obtained from the reference\(^5,6\). The component model has been validated against several experiments and shown to be able to simulate the nonlinear anchorage behavior quite well. However, due to the requirement of several springs to model a single anchor, the model becomes computationally expensive if it is used to consider the interaction between several anchorages with the structural members. In such cases, where a large number of anchorages are required to be simulated in a structural application, equivalent spring modelling approach offers a good solution. This is discussed in the next section.

### 3.2 Equivalent spring model

The equivalent spring model considers the nonlinear behavior of an anchorage through a single spring whose characteristics are derived either on the basis of the experiments or detailed numerical or component models. Sharma\(^7\) developed one such equivalent spring model to consider the nonlinear behavior of the anchorages used to connect the haunch elements to the reinforced concrete beam and column for the retrofitting of beam-column joints (Figure 4).
In this model, the load-displacement behaviour of the anchorage system is idealized in a penta-linear format as shown in Figure 5, which can reasonably well represent the typical load-displacement behaviour of the anchors. The coordinates of points A, B, C and D as shown in Figure 5 depend on the estimated failure mode for the anchor group. In case of anchors subjected to tension loads, the concrete cone failure is known to be most brittle while the steel failure is known to be most ductile. The pull out or pull through or bond failure fall in between concrete and steel failure in terms of ductility. This phenomenon can be accounted for in the model by varying the distance between points B and C as well as between points C and D in Figure 5. Certain assumptions were made to combine the stiffness of the individual anchors and arrive at the load-displacement behavior for the complete anchorage. The stiffness values for different types of anchors as well as the ratio of the stiffness of the anchors in cracked and uncracked concrete were based on the work by Mahrenholtz. It may be noted that similar models are also used to consider the interactions between the structure-anchorage-piping under seismic loads.

Figure 4: Equivalent spring model developed by Sharma for simulating the nonlinear behavior of the anchorage used in connecting the strengthening elements

Figure 5: Generalized idealized load-displacement behaviour used for the anchorage system in the equivalent spring model (Sharma et al.)
The equivalent spring model was validated against the tests on beam-column joints strengthened using fully fastened haunch retrofit solution (FFHRS), where the steel haunch elements were connected to the frame members using post-installed anchors\textsuperscript{7,12}. The numerical modeling of the joint was performed using the beam elements and the inelastic characteristics were modeled through springs within the framework of lumped plasticity approach as shown in Figure 4. The geometry of all the test specimens as well as the haunch element was essentially the same. Four different cases were tested, namely JT1-3, JT1-4, JT1-5 and JT1-6. To connect the haunch elements in case of JT1-3 and JT1-4 bonded anchors were used, for JT1-5 concrete screws were employed and for JT1-6 expansion anchors were used\textsuperscript{7,12}. In case of JT1-4, selective weakening was employed in addition to FFHRS and the middle beam bar was cut\textsuperscript{7,12} to reduce the beam yield capacity.

It is important to note that the load carrying capacity of the anchorage calculated using the force based methods was almost the same for all the anchorages irrespective of the type of anchor used and concrete cone failure was the expected failure mode in each case. Figure 6 shows the results of the tests in terms of the failure mode. It can be seen that even though the theoretical load carrying capacity for all the anchorages was very similar and the design of the joints and strengthening elements used in the tests was essentially the same, each test failed with a different failure mode.

In joint JT1-3, the first cracks were observed in the beam, at the shifted critical section i.e. the location of the farthest anchors in the haunch element but beyond the peak load, anchorage failure occurred in the form of concrete cone failure. In the case of JT1-4, due to selective weakening of the beam, the failure mode was due to the beam yielding, which was also the desired failure mode. In case of JT1-5 concrete cone failure occurred at the anchor group of concrete screws in the column, after which significant damage in the joint was observed. In case of JT1-6, the expansion anchors underwent large displacements and the force transfer between the structural members and the haunch element could not take place efficiently.

These test results clearly highlight that the actual performance of a strengthening connected using the anchorages depends largely on the performance of the anchorage itself. The current force based design and analysis methods are unable to account for this difference in the performance of the anchors. In order to ensure the safety of the structural strengthening solutions or to consider the interaction of various components and structure connected using anchorages, an approach that can give due consideration to the anchor performance in structural analysis is essential. The results

Figure 6: Failure modes obtained from different tests on beam-column joints strengthened using FFHRS\textsuperscript{7,12}
obtained by modeling the anchorage through an equivalent spring model by Sharma\textsuperscript{7} for the case of JT1-3 and JT1-4 are summarized in Figure 7. In order to consider the hysteretic behavior, Pivot hysteresis model was assigned to the nonlinear springs\textsuperscript{7}. More details and other results can be found in the reference\textsuperscript{7}. From Figure 7, it can be observed that the equivalent spring model is suitable to consider not just the load carrying capacity but the complete performance of the anchorage in the analysis.

![Figure 7: Comparison of experimental and numerical results obtained using the equivalent spring model\textsuperscript{7}](image)

4 Conclusions

This paper is an attempt to highlight the importance of considering the entire performance of the anchorages in analysis and design, which is so far neglected in the current force based methods. Several limitations of the force based design methods, e.g. consideration of stiffness of the anchorages, requirement of stiff base plate, assumption of having all the anchors of a group intercepted by a crack, no consideration of the re-distribution of forces among the anchors etc. are discussed. The general concept of the performance based approach for the structures and its extension to the anchorages is presented. The component model and the equivalent spring model that can be used for the performance assessment of the anchorages are given. It is emphasized through an example of strengthening of beam-column joints using FFHRS that the displacement behavior of the anchors has a significant influence on the performance of the anchorage used in seismic strengthening even though the load carrying capacity of the anchorage is the same. This shows that the force based design methods for the anchorages might not only be inadequate but also unreliable in certain cases. It is further shown that simple models such as equivalent spring models can duly and reasonably consider the performance of the anchors in structural analysis.

References:


5. Kuhlmann et al., “New Market Chances for Steel Structures by Innovative Fastening Solutions between Steel and Concrete (INFASO)” final report, RFSR-CT-2007-00051, 2010


SEISMIC ANCHOR PERFORMANCE CATEGORIES AND PERFORMANCE BASED DESIGN

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ABSTRACT

Performance based design (PBD) is an established design philosophy in earthquake engineering. It allows the seismic structural design of buildings and bridges based on defined performance levels via an economic approach to overcome predicted uncertainties of the seismic demands. In a similar fashion, seismic anchor design is also possible for various performance levels, termed categories C1 and C2, for concrete anchors used in structural and nonstructural applications. However, design engineers are often confused about the use of these performance categories and their potential implications on PBD.

The goal of this paper is to clarify the idea of “performance” in PBD and seismic anchor design as well as their correlation. The background of PBD as well as of seismic anchor design is briefly outlined and anchor performance is discussed in regard to design and product qualification in greater detail. This includes an explanation of the seismic anchor performance categories and why qualification for C2 is more demanding than C1. The paper strives to help engineers understand the current requirements for a seismic design using C1 or C2 anchors as well as will deliver an important contribution to the discussion on the current design requirements in the context of building behavior.

1 Introduction

In engineering design, determination of demands is a vital task in the selection of material properties and structural element geometry. For the design professional, it is a critical matter of liability to follow the accepted and governing codes of practice. Safety and failure prevention are the key targets of this exercise, but serviceability considerations contribute significantly to minimizing economic losses. Within earthquake engineering, performance based design (PBD) is chosen to select target performance criteria for a particular structure (Fig. 1a).
In addition to design of structures, concrete anchors in safety relevant applications (e.g., nuclear power plants and other facilities) also require professional engineering design. During the service life, which is generally assumed as 50 years for structures and anchors installed therein, anchors are exposed to various loading types and environmental conditions. These conditions include seismically induced cyclic loading within significant cracking in the concrete base material. These conditions are represented in the test conditions used to qualify anchors for the intended application and to determine their performance characteristics which are then used for a professional design (Fig. 1b). In case of seismic design, the European design code permits anchorage design within two different seismic performance categories, namely C1 and C2. However, the understanding of the term performance is different in PBD and anchor design, resulting in confusion and an incorrect application of performance based design for seismic anchorages.

2 Background

2.1 Structural Design and Anchor Design

In Europe, the introduction of the European building code series (Eurocode - EC) established a load and resistance factor design (LRFD) based on a partial safety concept. The LRFD allows an analyst to allocate specific safety factors ($\gamma$) to both design forces and the material resistance strengths for a defined failure probability. The failure probability is in general defined as $10^{-6}$ (EN 1990), however this may be adjusted via National Annexes. Appropriate design safety factors for the ultimate limit state design (ULS), as described in Section 3.3 in EN 1990, ensures the structural reliability. For the serviceability limit state design (SLS), as described in Section 3.4 in EN 1990 under realistic conditions, the safety factors are generally decreased to 1.0 (Table 1). An example for an SLS design is to limit deflection of concrete elements according to EN 1992. In similar fashion, displacement design data of concrete anchors designed according to Part 4 of EN 1992 may be used to verify anchor displacement (Section 11 in EN 1992 Part 4). However, it is noteworthy that anchor design for ULS is strictly force-based, where displacement controlled failures are outside of the scope of the current design code.
Table 1: Overview design methods in Europe and the United States

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Partial Safety Factor Design</th>
<th>Strength Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Serviceability (SLS)</td>
<td>Eurocode</td>
<td>Load (force)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>( \gamma_F = 1.0 )</td>
</tr>
<tr>
<td>Ultimate (ULS)</td>
<td></td>
<td>Factored</td>
</tr>
<tr>
<td></td>
<td></td>
<td>e.g. ( \gamma_M = 1.15 ) for steel, ( \gamma_M = 1.5 ) for concrete</td>
</tr>
</tbody>
</table>

In the United States, the structural concrete design code ACI 318\(^4\) proposes strength design for concrete elements for their ultimate capacity. To compute the design resistance, the nominal strength is multiplied by the corresponding strength reduction factors \( \phi \). Likewise, the design load is derived using factored load combinations as defined in ASCE 7\(^5\) where the serviceability requirements are then outlined in Chapter 24 of ACI 318. Reduction factors are not used herein and according to Appendix C of ASCE 7 in the discussion for load combinations, unfactored loads represent realistic deflections at the service state. However, anchor displacement design is unknown within ACI 318, though e.g. underlying creep displacement effects may be critical for the strength data used for the design.

### 2.2 Earthquake Engineering and PBD

In earthquake engineering a serviceability requirement is often not specified; however, there is a damage limitation requirement (DLR). In European design terminology, this equates to a damage limitation state (DLS), while a non-collapse requirement (NCR) is designed for the ULS according to EN 1998\(^6\). DLS and ULS are associated with different earthquake probabilities and associated return periods. Specifically, this is a 10\% probability of exceedance (PoE) in 10 years (95 year return period) and a 10\% probability in 50 years (475 year return period) for DLS and ULS, respectively. The rarer and more intense earthquake underlying the ULS design is often termed as design based earthquake (DBE). In the United States, seismic design loads according to ASCE 7-10 are first based on a yet stronger and even more rare, Maximum Considered Earthquake (MCE) with a 2\% probability in 50 years (2475 year return period). In the design process, the acceleration is often reduced to 2/3 of the nominal acceleration. This acceleration reduction is performed to approximately correspond to a DBE event with 475 year return period. ASCE 7-10 does not explicitly define requirements for damage limitations or collapse prevention, however in its absence it is reasonable to assume that the DLS design is also based on the MCE event. Both design codes of EN 1998 and ASCE 7 use importance factors that increase the design loads for critical buildings to improve their performance. Table 2 compiles common earthquakes design levels used in earthquake engineering.
As earlier discussed, modern building codes provide a high level of safety based on probabilistic considerations. However, earthquakes are very unpredictable and do not follow the codes. Therefore, it is not economical to construct buildings that will resist extreme earthquake loads without damage. The level of damage that will be sustained is a function of the performance criteria that the building is designed to meet, common to a PBD framework. While PBD methodology is not limited to earthquake engineering, its application in this context, is most prominent and closely aligned. The fundamental idea is that performance criteria are set which allow to achieve the best possible balance between construction costs and ultimate performance. There are several PBD guidelines available with various definitions (e.g. SEAOC\(^1\), FEMA 273\(^2\)) but in general performance levels are targeted for various earthquake frequencies (or return periods). With an increase in the building importance or occupancy (e.g. emergency critical facility versus an office building), an increased level of performance is desirable as outlined in Fig. 1a. For the basic objective of ordinary buildings (e.g. low rise residential buildings), the specific earthquake frequencies and thus probabilities and generally associated performance levels are shown in Table 2. For clarity, it is noted that the ground acceleration, in earthquake engineering a common measure for earthquake shaking at a particular site, increases with decreasing probabilities of exceedance (earthquake frequency). Or in other words, the ground acceleration increases with an increasing return period.

### Table 2: Design earthquake characteristics for ordinary building design (e.g., office building)

<table>
<thead>
<tr>
<th>Earthquake frequency</th>
<th>Return period (yr)</th>
<th>PoE(^1)</th>
<th>Earthquake category</th>
<th>Limit state</th>
<th>Requirement</th>
<th>Performance level for basic objective</th>
<th>Goal</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequent</td>
<td>43</td>
<td>50% in 30 years</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Fully operational / or functional</td>
<td>No significant damage to structural and nonstructural elements</td>
<td></td>
</tr>
<tr>
<td>Occasional</td>
<td>72</td>
<td>50% in 50 years</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Immediate occupancy / operational</td>
<td>No significant damage to structural elements; nonstructural elements are secure and most would remain operational</td>
<td></td>
</tr>
<tr>
<td></td>
<td>95</td>
<td>10% in 10 years</td>
<td>-</td>
<td>DLS (EC)</td>
<td>DLR – damage limitation</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>237</td>
<td>10% in 25 years</td>
<td>OBE – Operating Basis EQ(^2)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Rare</td>
<td>475</td>
<td>10% in 50 years</td>
<td>DBE – Design Basis EQ</td>
<td>ULS (EC)</td>
<td>NCR – non-collapse</td>
<td>Life safety</td>
<td>Significant damage to structural elements; nonstructural elements are secure but not functional</td>
<td></td>
</tr>
<tr>
<td>Very rare</td>
<td>970</td>
<td>10% in 100 years</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Collapse prevention</td>
<td>Substantial damage to structure and nonstructural elements</td>
<td></td>
</tr>
<tr>
<td>Extremely rare</td>
<td>2475</td>
<td>2% in 50 years</td>
<td>MCE – Maximum Considered EQ</td>
<td>2/3 of acceleration used for design (USA)</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\(^1\) PoE = Probability of exceedance; EQ = Earthquake; EC = Eurocode

\(^2\) Dependent on the structure considered, 10% in 25 years often defined as OBE for nuclear facilities and other critical structures
3 Performance Considerations in Anchor Technology

3.1 Establishing Anchor Performance

In the context of concrete anchors, EN 1992 and ACI 318 uses the term *performance* in the sense of strength and displacement characteristics. These performance data are provided in approval documents (ETA in Europe and ICC-ES in the United States) and allow an engineered anchor design. Qualification guidelines (EAD 330232-00-0601, EAD 330499-00-0601, ACI 355.2, ACI 355.4) define the test program an anchor seeking approval has to undergo. The tests can be broadly categorized as admissible service condition tests (*serviceability tests*) and severe conditions tests for robustness (*suitability tests*). Serviceability tests are run under realistic conditions, in particular where the characteristic crack widths represent normal conditions. For ordinary environmental expositions, EN 1992 and its National Annexes, establishes 0.3 mm as the maximum allowable crack width for characteristic loads. For suitability tests representing adverse conditions, the maximum crack width is increased to 0.5 mm. This crack width roughly corresponds to the upper fractile value, herein defined as 95% at 90% confidence level (Table 3). However, it should be noted that performance data associated with service condition and derived from serviceability tests are not necessarily related to the design for SLS nor may the performance derived from suitability tests be equalized with the design strength for ULS.

Table 3: Some critical parameters for anchor qualification testing

<table>
<thead>
<tr>
<th>Qualification test type</th>
<th>Test condition</th>
<th>Example parameter: crack width</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Static and seismic C1</td>
</tr>
<tr>
<td>Serviceability tests</td>
<td>Realistic test conditions</td>
<td>0.3 mm</td>
</tr>
<tr>
<td>Suitability tests</td>
<td>Adverse test conditions</td>
<td>0.5 mm</td>
</tr>
</tbody>
</table>

3.2 Importance of Anchors for Achieving Performance Levels

Anchor performance plays a vital role in achieving various building performance objectives. For structural anchorages, the primary goals are life safety and collapse prevention via a sufficient ultimate load capacity. For anchorages of nonstructural components, operational functionality and immediate occupancy are critical performance levels to be achieved through damage prevention. While the load demand may be lower than for collapse prevention, vulnerable nonstructural components may be rendered inoperable at much lower displacement demands than those acceptable for collapse prevention. This is a result of the displacement sensitivity of many common nonstructural components. The table in Fig. 2 is taken from FEMA 273 and defines reasonable overall building performance levels for a matrix of structural and nonstructural performance levels.
4 Seismic Anchor Performance Categories in Europe

4.1 C1 and C2 Seismic Anchors

Seismic anchor performance categories C1 and C2 have been first introduced with the seismic amendment Annex E of the ETAG 001\textsuperscript{12} in 2013. C2 category was developed by an international research team and is based on methodological investigations of seismological input parameters, nonlinear building simulations including the response of nonstructural components, and experimental verification tests which involved simulated seismic tests on anchors as well as large scale shake table tests. To allow direct comparability and to align with internationally recognized standard ACI 355, the ETAG 001 Annex E included also the C1 category which is quite similar to the seismic anchor qualification testing according to ACI 355. Since the test conditions used for C2 qualification are more demanding than those used for C1 qualification, C2 qualification generally results in reduced performance and thus in lower design strengths in comparison to C1 qualification.

ETAG 001 Annex E presents a table relating the seismic performance categories C1 and C2 to the seismic hazard level and building importance classification. The level of seismicity is a function of $a_g S$, where $a_g$ is the design ground acceleration on Type A ground (rock) and $S$ the soil factor in accordance with EN 1998. The local design ground accelerations can be found in seismic hazard maps of the EN 1998 National Annex and are generally based on the DBE (10% probability in 50 years). Part 4 of EN 1992 shows the same table (Fig. 3). It is noted that the assignment of the seismic performance categories C1 and C2 to the seismicity level and building importance classes may be redefined in the National Annex of the EN 1992. For clarity it is noted here that in earthquake engineering the term seismicity is generally understood as the geographical and historical distribution of earthquakes in a particular region, whereas the ground acceleration is understood as a common intensity measure at a specific site of interest.
The innovative element within the C2 qualification is the testing and evaluation procedures related to serviceability and suitability levels, similar to static qualification testing, and the direct link to DLS and ULS. Moreover, testing on serviceability and suitability level can be conducted simultaneously through the use of a single test protocol on the same test specimen. Note this is a single protocol for each type of loading, namely: tension load cycling, shear load cycling, and crack cycling (example in Fig. 4a). In application of ACI 355 seismic qualification, i.e. C1 category, test results only provided a reduction factor to calculate seismic design strength based on the static design strength. In contrast, the C2 qualification provides design data including seismic design strength as well as anchor displacement for the damage prevention design at DLS and the anticipated worse case displacement at ULS. For this test protocol, the maximum crack width is increased in comparison to static qualification testing (refer to Table 3). This is done to permit the unrestricted application of C2 anchors in all areas outside of zones of plasticity, for which a maximum crack width of 0.8 mm is assumed. In contrast, anchors with a less strict C1 qualification are limited to installations where the crack widths are equal or less than 0.5 mm.

### Table C.1 — Recommended seismic performance categories for fasteners

<table>
<thead>
<tr>
<th>Seismicity level&lt;sup&gt;a&lt;/sup&gt;</th>
<th>Importance Class acc. to EN 1998–1:2004, 4.2.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Class</td>
<td></td>
</tr>
<tr>
<td>2 Very Low&lt;sup&gt;b&lt;/sup&gt;</td>
<td>$a_d \cdot S \leq 0.05g$</td>
</tr>
<tr>
<td>3 Low&lt;sup&gt;b&lt;/sup&gt;</td>
<td>$0.05g &lt; a_d \cdot S \leq 0.1g$</td>
</tr>
<tr>
<td>4 &gt; low</td>
<td>$a_d \cdot S &gt; 0.1g$</td>
</tr>
</tbody>
</table>

<sup>a</sup> The values defining the seismicity levels are subject to a National Annex. The recommended values are given here.

<sup>b</sup> Definition according to EN 1998–1:2004, 3.2.1.

<sup>c</sup> $a_d$ = design ground acceleration on type A ground (see EN 1998–1:2004, 3.2.1).

<sup>d</sup> $S$ = soil factor (see EN 1998–1:2004, 3.2.2).

<sup>e</sup> C1 for fixing non-structural elements to structures (Type B connections).

<sup>f</sup> C2 for fixing structural elements to structures (Type A’ connections).

#### 4.2 Qualification and Design Data

The innovative element within the C2 qualification is the testing and evaluation procedures related to serviceability and suitability levels, similar to static qualification testing, and the direct link to DLS and ULS. Moreover, testing on serviceability and suitability level can be conducted simultaneously through the use of a single test protocol on the same test specimen. Note this is a single protocol for each type of loading, namely: tension load cycling, shear load cycling, and crack cycling (example in Fig. 4a). In application of ACI 355 seismic qualification, i.e. C1 category, test results only provided a reduction factor to calculate seismic design strength based on the static design strength. In contrast, the C2 qualification provides design data including seismic design strength as well as anchor displacement for the damage prevention design at DLS and the anticipated worse case displacement at ULS. For this test protocol, the maximum crack width is increased in comparison to static qualification testing (refer to Table 3). This is done to permit the unrestricted application of C2 anchors in all areas outside of zones of plasticity, for which a maximum crack width of 0.8 mm is assumed. In contrast, anchors with a less strict C1 qualification are limited to installations where the crack widths are equal or less than 0.5 mm.

Figure 4: a) Example of seismic test protocol for serviceability and suitability level testing; b) Performance level matrix according to ASCE 7 (RC = risk category)
4.3 Anchor Performance Data and PBD

In summary, EN 1992 Part 4 requires C2 qualified anchors for buildings with higher importance and/or at higher design ground accelerations. In terms of performance, C2 anchors must be used when C1 anchors are insufficient to meet the required performance level (refer to Fig. 3). However, the superior performance of C2 anchors is not expressed by better design parameters, such as increased design strength and decreased design displacement, but rather verified by a more rigorous test regime accounting for more adverse conditions. It is important to note that the test parameters (e.g. crack width) not only depend on the demand imposed by the earthquake intensity (i.e. ground acceleration which is a function of the earthquake return period), but also on the dynamic response characteristics of the building. Moreover, acceleration induced earthquake forces are not an input parameter for the anchor qualification, but only considered in an ULS anchor design. Anchor displacement data (at DLS and ULS levels) permit a secondary design check, however EN 1992 Part 4 identifies that anchor displacements smaller than what is specified in the ETA as characteristic may be required. This note is related to the anchorage application of displacement sensitive nonstructural components or rigid supports.

Performance levels in PBD are variable and mostly displacement focused, where the increased performance is measured by reduced structural deformation, e.g. decreased interstory drift or reduced concrete degradation and therefore smaller crack widths. For structural design codes such as the EN 1998 or ACI 318, however, the performance level is fixed to damage limitation (DLS) and collapse prevention (ULS). ASCE 7 briefly addresses building performance for different risk categories (akin to the building importance factor) and ground motion in the fashion of PBD, but with a different intonation (Fig. 4b). ASCE 7 differentiates the damage prevention state further, and requires for above-ordinary buildings a stricter MCE performance level beyond that of collapse prevention.

5 Regulative Status for C1/C2 Requirement

Since the introduction of the Annex E of the ETAG 001 qualification guideline in Europe, numerous anchors have been qualified for C1 and C2 categories. Part 4 of EN 1992 has been officially introduced throughout Europe in January 2017. As a result of this action, the dualism of seismic anchors for either C1 or C2 performance category became a design standard. The table defining the requirements for C1 or C2 design (Fig. 3) was very much debated. In particular, the assignment of the performance categories C1 and C2 merely to the seismicity level and building importance classes, but not to the actual installation condition (e.g. of the base material in view of crack width) was criticized. However, in current design practice e.g. for anchorages in nuclear power plants (RCC-CW13), the choice of anchor performance category is solely based on the actual design crack width. In this sense, the revision made in German National Annex to EN 1992 recommends C1 and C2 anchors primarily based on the crack width. Specifically this is C1 for maximum crack width of 0.5 mm and C2 for maximum crack width of 0.8 mm. In these discussions, anchor performance was the primary discussion topic, where PBD was not included.
6 Summary and Conclusion

Concrete anchors in Europe can be qualified and designed for two seismic anchor performance categories. The present paper reviews critical aspects of how anchor performance ties into performance based design (PBD).

- Where anchors connect structural or nonstructural elements within a concrete structure, the anchor performance has a direct impact on the structural and nonstructural performance of the building and its contents and functionality. This holds for the damage limitation state as well as for the ultimate load state.

- Current US and European structural design codes for reinforced concrete, including concrete anchor design, only reflect PBD principles rudimentarily through different design levels to meet collapse prevention or damage limitation requirements, as well as by the building importance classes. However, a design approach for anchor displacement as primary design target is lacking for a displacement focused PBD.

- Seismic anchor performance categories C1 and C2 are founded in specific test conditions and not within the performance objectives. Due to the more demanding test conditions including a larger crack width, C2 qualification is generally more demanding in comparison to C1 qualification and therefore superior.

- Requirements for anchorage design within the C1 and C2 categories currently depend on building importance class and ground acceleration. It is debatable whether design acceleration for a target anchorage design would be more meaningful, rather than the ground acceleration taken directly from seismic hazard map. Moreover, the crack width, a parameter that is critical for the anchor performance and that varies between the categories C1 and C2, should be taken into account when determining the performance category to be used for the specific design.

The views expressed in this paper are the views of the authors only and do not necessarily reflect the views of the authors’ affiliations of Stanley Black & Decker, Inc. nor the University of Nebraska-Lincoln.

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NUMERICAL INVESTIGATIONS ON CIRCULAR ANCHORAGES WITH HEADED STUDS LOADED IN TENSION OR SHEAR TOWARDS THE EDGE

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ABSTRACT

The current design provisions for anchorages include single anchors or anchor groups with anchors arranged in a rectangular pattern1,2. However, due to architectural or functional reasons such as accessibility, anchorages other than rectangular are also used. One such popular anchorage type is the anchors arranged in a circular manner. Currently, no explicit guidelines exist for such anchorages, and the design is done primarily on the basis of engineering judgement or experience.

This paper presents the results of preliminary numerical investigations carried out on a group of headed studs positioned in a circular arrangement to study their behavior and performance under tension and shear loading. For this study, anchorages with eight headed studs arranged equidistant from each other on a circular anchor plate are taken into consideration and a 3D finite element analysis is carried out using the software MASA³.

The anchorages are positioned at three different edge distances from the concrete edge, towards which the anchorage is loaded in shear. The numerical investigations show that under the shear loads towards the edge, the failure crack appears to be originating from the back anchor row. Under tension loads, the anchorage fails by forming a concrete cone similar to the rectangular groups and the results suggest that a method similar to the concrete cone design (CCD method)⁴ given in current codes could give a reasonable solution to calculate failure loads for circular anchorages under tension loads as well. In future studies, experimental investigations will be carried out to validate the numerical results and to develop a design concept for circular shaped anchorages subjected to tension or shear loads.

1 Introduction

Anchorages using multiple headed studs are known to form one of the most reliable types of fastenings that may perform well even in unfavorable conditions such as cracked concrete with relatively large crack widths and seismic loads⁵. Over the past few decades, large amount of research has been carried out on the anchorages under tension loads, shear loads as well as interaction loads. A significant portion of this research work, which also had a strong influence on the development of standards and guidelines, has been done at the University of Stuttgart, which has been largely
summarized by Eligehausen et al.\textsuperscript{6}. During the last decade, the research work on anchorages has also been extended to cover the aspects of seismic loads\textsuperscript{7,8,9}.

Based on the research work, the design guidelines have been developed such as EN1992-4\textsuperscript{1} and fib Bulletin 58\textsuperscript{10} in Europe and ACI 318\textsuperscript{2} in the US. However, the scope of the current design standards is limited to anchorages with anchors arranged in regular and rectangular configuration. Furthermore, the number of anchors in a row and the number of anchor rows are also limited by the standards, e.g. EN1992-4 and fib Bulletin 58 limit the design rules to be valid for the configurations shown in Figure 1.

![Figure 1: Anchor configurations cover by EN1992-4 and fib Bulletin 58](image)

Under tension loads for concrete cone failure, the design rules are primarily based on the concrete capacity method\textsuperscript{4}. For anchorages close to an edge subjected to shear forces towards the edge, the design rules to determine the concrete edge failure load are based on the work of Hofmann (2004)\textsuperscript{11}. However, for the anchorages with multiple anchor rows close to an edge, the design approach of EN1992-4 and the fib Bulletin 58 to determine concrete edge failure load differs in the consideration of the failure crack. While EN1992-4 assumes that the failure crack for the anchorages with more than one anchor row parallel to the edge appear from the front anchor row, fib Bulletin 58 assumes that the failure crack for such anchorages appear from the back anchor row. The implication of assuming the failure crack appearing from the front anchor row has been highlighted by Sharma et al. (2017)\textsuperscript{9} and explained in Figure 2.

![Figure 2: Current assumption of design failure for concrete edge failure according to EN1992-4\textsuperscript{9}](image)

a) Group with one anchor row (1x2 group)  
b) Group with 3 anchor rows (3x2 group)

This approach leads to no difference between the groups having a single anchor row or multiple anchor rows parallel to the edge is the edge distance of the front anchor row is the same. However,
the tests performed by Grosser (2012)\textsuperscript{12} and Sharma et al. (2017)\textsuperscript{9} clearly show that this approach is overly conservative and in reality the failure crack always appears from the back anchor row as shown in Figure 3.

So far, all the research work has been performed on anchorages with anchors arranged in a regular rectangular pattern. The concrete capacity design method for tension loads is also valid for rectangular anchor groups and the guidelines for consideration of the failure crack in case of concrete edge failure are also available only for rectangular anchor groups. However, in practice, due to architectural or functional reasons or due to the accessibility, anchorages other than rectangular are also used. One such popular anchorage type is the anchors arranged in a circular manner (Circular anchor group). Circular anchor groups are commonly used for cylindrical columns, lighting poles, electric power line posts and to make end pipe connections in the industrial buildings\textsuperscript{13, 14}. Figure 4 shows one such application of the circular anchor groups to connect the concrete filled steel tubes to the reinforced concrete foundation\textsuperscript{15}.

Currently, no explicit guidelines exist for such anchorages and the design is done based on engineering judgement\textsuperscript{10, 16}. In the fib Task Group TG 2.9 “Fastenings to structural concrete and masonry”, it is being discussed that in order to have a uniform safety in the design of anchorages, the rules for anchorage design should be extended to cover circular anchor groups as well. However, there are currently no test results that could be used for development of design rules for circular anchor groups.

This paper examines the behavior of circular anchor groups under tension and shear loading through 3D finite element analysis.
2 Numerical Analysis

The 3D finite element (FE) model is developed using the pre- and post-processing commercial software FEMAP\textsuperscript{17} and the nonlinear analysis is performed using the program MASA\textsuperscript{3} (MAcroscopic Space Analysis) developed at the University of Stuttgart.

2.1 Geometry of the anchor plate

Anchorages with eight headed studs are taken into consideration. Each of the headed studs is arranged equidistant from each other and considered to be perfectly welded on a circular anchor plate. The load is applied in the middle of the anchor plate. The direction of the shear load was towards the edge of the concrete slab. The test program followed is given in Table 1.

<table>
<thead>
<tr>
<th>Test series</th>
<th>Slab height [h] [mm]</th>
<th>Edge distance [c_1] [mm]</th>
<th>Type of load</th>
<th>Position of the load</th>
<th>Cylindrical Compressive Strength [f_{ck}] [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>450</td>
<td>100</td>
<td>Shear</td>
<td>middle</td>
<td>25</td>
</tr>
<tr>
<td>2</td>
<td>450</td>
<td>200</td>
<td>Shear</td>
<td>middle</td>
<td>25</td>
</tr>
<tr>
<td>3</td>
<td>450</td>
<td>300</td>
<td>Shear</td>
<td>middle</td>
<td>25</td>
</tr>
<tr>
<td>4</td>
<td>1000</td>
<td>100</td>
<td>Shear</td>
<td>middle</td>
<td>25</td>
</tr>
<tr>
<td>5</td>
<td>1000</td>
<td>200</td>
<td>Shear</td>
<td>middle</td>
<td>25</td>
</tr>
<tr>
<td>6</td>
<td>1000</td>
<td>300</td>
<td>Shear</td>
<td>middle</td>
<td>25</td>
</tr>
<tr>
<td>7</td>
<td>1000</td>
<td>500</td>
<td>Tension</td>
<td>middle</td>
<td>24</td>
</tr>
</tbody>
</table>

The outer diameter of the circular plate is considered as 580 mm while the headed studs are arranged along the circle of diameter 500 mm on the plate. The headed studs diameter is taken as \(d_s = 22\) mm. Under shear loading, the anchor plate is positioned with three different edge distances, \(c_1\), namely 100 mm, 200 mm and 300 mm. On the contrary, for tension loading, the distance from the concrete edge is taken as 500 mm to avoid the influence of edge on the concrete cone breakout. Two different slab heights are studied, \(h = 450\) mm and \(h = 1000\) mm. The thickness of the base plate is considered as 30 mm and the effective embedment depth of the anchors is taken as \(h_{ef} = 150\) mm. The anchorage investigated is illustrated in Figure 5.
2.2 Modeling parameters
The headed studs and the base plate were modelled using 8-node hexahedral elements (Figure 5), while the concrete was meshed using four node tetrahedral elements. The contact between the steel plate or headed studs and surrounding concrete was modelled using bar elements that do not take up any tension forces. The nonlinear material behaviour of concrete was modelled with Microplane model for concrete with relaxed kinematic constraint. A linear elastic behaviour of the anchors and the base plate steel is assumed.

2.3 Material properties
The cubic compressive strength of concrete is assumed as $f_{cu} = 30.0$ MPa, the tensile strength of concrete is taken as $f_t = 2.50$ MPa and the fracture energy is considered as $0.07$ N/mm. The elastic modulus for concrete is taken as $E_c = 23500$ MPa and the Poisson’s ratio as, $\nu = 0.18$. The Young Modulus and Poisson’s Ratio for the headed studs as well as for the anchor plate is set to $200\,000$ MPa and 0.33 respectively.

2.4 Boundary and load conditions
The concrete slab was supported to allow a clear formation of the concrete edge under shear loads. Steel plates are introduced in the front and back side of the block and confined in horizontal direction (y-direction), the upper back edge of the concrete block was confined in two direction (x and z direction) to prevent uplift or lateral sliding of the slab (Figure 6). For the example under tension loads, all the upper edges of the concrete slab were constrained in the vertical direction. The load was applied in displacement control on the nodes in the middle on the top surface of the anchor plate.

Figure 6: Boundary conditions applied on the concrete slab for investigations under shear loads

3 Numerical simulations results

3.1 Anchor group under shear loads
Figure 7 to Figure 9 illustrate the failure mode and the load – displacement distribution in the concrete slabs with heights 450 and 1000 mm and an edge distance respectively of $c_1 = 100$ mm, $c_1 = 200$ mm, $c_1 = 300$ mm.
The numerical analysis reveals that with increasing the edge distance from 100 mm to 200 mm, the failure load increases between 10 to 12 % for both heights. Increasing the distance further to 300 mm shows barely an influence on the failure load, for the slab height of 450 mm, in comparison with the increase of 10 % in the slab with 1000 mm of height.

Figure 7: Crack distribution and load-displacement curve for $c_1 = 100$ mm
Figure 8: Crack distribution and load-displacement curve for $c_1 = 200$ mm

Figure 9: Crack distribution and load-displacement curve for $c_1 = 300$ mm

As it seen in the above figures, the numerical investigation shows, that in each case the failure of the structure starts from the previous to back row of the anchor plate. The break out bodies, for $h = 450$ mm are truncated because of the height of the concrete slab, whereas for $h = 1000$ mm the concrete cone is completely formed.
3.1.1 Comparison with EN1992-4¹

The characteristic resistance of a group of fasteners loaded towards the edge as given in EN1992-4¹ is:

\[ V_{Rk,c} = V_{Rk,c}^0 \frac{A_{c,V}}{A_{c,V}^0} \psi_{s,V} \psi_{h,V} \psi_{e,c,V} \psi_{a,V} \psi_{r,e,V} \]  

(1)

The initial resistance of a single fastener loaded perpendicular to an edge is given in Equation (2):

\[ V_{Rk,c}^0 = 2.4 \times d_{nom}^a \times l_f^\beta \overline{f_{ck}} \times c_1^{1.5} \]  

(2)

Where:

\[ d_{nom} \] = the diameter of the headed stud

\[ l_f = h_{ef} \] in case of a uniform diameter of the shank of the headed stud

\[ \alpha = 0.1 \times \left( \frac{l_f}{c_1} \right)^{0.5} \]

\[ \beta = 0.1 \times \left( \frac{d_{nom}}{c_1} \right)^{0.2} \]

\[ A_{c,N}^0 \] is the reference projected area (Equation (4)) and \( A_{c,N} \) is the actual projected concrete cone area on the specimen as shown in Figure 3.3.

\[ A_{c,N}^0 = 4.5 \times c_1^2 \]  

(4)

Figure 10: Actual projected area \( A_{c,N} \) for a group of anchor according to EN1992-4¹

According to EN1992-4¹, \( V_{Rk,c}^0 \) is the 5% fractile of the resistance, hence it should be multiplied with 1.33 in order to get the mean value which afterwards will be compared with the results of the simulations.

In Equation (1):

\[ \psi_{s,V} \] considers the disturbance of the distribution of stresses in the concrete due to the edge distance.

\[ \psi_{h,V} \] takes into account the member thickness. For \( h = 1000 \) mm is taken as equal to one, however for \( h = 450 \) mm different values of \( \psi_{h,V} \) are calculated depending on the concrete edge distance \( c_1 \). The factor is given with the formula:
\[
\psi_{h,V} = \left( \frac{1.5 \cdot c_1}{h} \right)^{0.5}
\]  
(5)

\(\psi_{e,c,V}\) takes into account the group effect when different shear loads are acting on the individual fasteners of a group.

\(\psi_{\alpha,V}\) considers the inclination of the shear load to the edge.

\(\psi_{r,e,V}\) takes into account the effect of the reinforcement located on the edge.

All the \(\psi\)- factors are taken as equal to 1 with the exception of \(\psi_{h,V}\) for \(h = 450\) mm.

Table 2 shows the shear load calculations assuming the failure crack starts from the 1\(^{st}\) row, previous to last row and last row.

<table>
<thead>
<tr>
<th>Edge distance (c_1)</th>
<th>(V_{u,c1})</th>
<th>(V_{u,cn})</th>
<th>(V_{u,cn-1})</th>
<th>(V_{u,FE})</th>
<th>(V_{u,FE} / V_{u,c1})</th>
<th>(V_{u,FE} / V_{u,cn})</th>
<th>(V_{u,FE} / V_{u,cn-1})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(h = 450) mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>58.4</td>
<td>250.7</td>
<td>274.2</td>
<td>384.8</td>
<td>6.6</td>
<td>1.5</td>
<td>1.4</td>
</tr>
<tr>
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<td>99.7</td>
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<td>309.6</td>
<td>431.4</td>
<td>4.3</td>
<td>1.5</td>
<td>1.4</td>
</tr>
<tr>
<td>300</td>
<td>138.9</td>
<td>322.8</td>
<td>344.8</td>
<td>430.0</td>
<td>3.1</td>
<td>1.3</td>
<td>1.2</td>
</tr>
<tr>
<td><strong>Mean Value</strong></td>
<td><strong>4.7</strong></td>
<td><strong>1.5</strong></td>
<td><strong>1.3</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(h = 1000) mm</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>100</td>
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<td>1.3</td>
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<td>536.7</td>
<td>647.1</td>
<td>4.7</td>
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<td>1.2</td>
</tr>
<tr>
<td><strong>Mean Value</strong></td>
<td><strong>9.0</strong></td>
<td><strong>1.3</strong></td>
<td><strong>1.3</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 11 illustrates the relation between the FE-computation for \(c_1 = 300\) mm and \(h = 1000\) mm, and the calculations according to EN1992-4\(^{1}\). As it can be seen, EN 1992-4\(^{1}\) is slightly conservative in comparison to the FE results. In order to further compare, we need experimental confirmation.
3.1.2 Anchor plate under tension load

The anchor plate under tension load fails similar to the rectangular anchor groups by forming a concrete cone. Figure 12 shows the crack propagation in the concrete specimen as well as the load-displacement curve.

![Figure 12: Crack propagation and load-displacement curve for the anchor plate loaded in tension](image)

3.1.3 Comparison with EN1992-4

The characteristic concrete cone capacity for an anchor group in tension, according to EN1992-4 is given by the formula:

\[ N_{Rk,C} = N_{Rk,C}^0 \frac{A_{c,N}}{A_{c,N}^0} \psi_{s,N} \psi_{f,e,N} \psi_{e,c,N} \psi_{M,N} \]  \hspace{1cm} (1)

\[ N_{Rk,C}^0 = k_{ucr} \sqrt{f_{ck}} h_{ef}^{1.5} \]  \hspace{1cm} (2)

\( N_{Rk,C}^0 \) is the characteristic resistance of a single anchor in non-cracked concrete and is given by the formula:

\[ N_{Rk,C}^0 = k_{ucr} \sqrt{f_{ck}} h_{ef}^{1.5} \]  \hspace{1cm} (2)

\( k_{ucr} \) is a factor which takes into account the influence of load transfer mechanisms for applications in non-cracked concrete and is equal to 12.7 for cast-in headed studs, \( f_{ck} \) is the cylindrical compressive strength of the concrete and \( h_{ef} \) is the embedment depth of the headed stud. As explained in section 3.1.1, the characteristic resistance \( N_{Rk,C}^0 \) should be multiplied with 1.33 in order
to get the mean value. By substituting the values of $k_{u cr}, f_{ck}, h_{ef}$, we get the mean resistance of a single anchor of 152,0 kN. $A_{c,N}^0$ is the reference projected area and $A_{c,N}$ is the actual projected concrete cone area on the specimen. Equation (3) gives the formula to calculate the reference area:

$$A_{c,N}^0 = s_{cr,N} * s_{cr,N}$$

where $s_{cr,N}$ is the spacing between the anchors and is taken as 3 $h_{ef}$.

There are two possibilities to calculate the actual area $A_{c,N}$. Figure 13 illustrates the expected concrete cone breakout of the circular plate in tension assuming the entire rig comes out whereas in Figure 14, the concrete break out is limited to 1.5 $h_{ef}$ inside and outside the anchor plate. From these two cases, the first one, namely when the entire rig comes out, shows better comparison to the code.

Consequently, the actual area $A_{c,N}$ is calculated as given in Equation (5):

$$A_{c,N} = \frac{\pi}{4} * (3h_{ef} + D)^2$$

In Equation (1), the factor $\psi_{s,N}$ takes into account the distribution of stresses due to the edge distances in concrete, $\psi_{re,N}$ takes into account the shell spalling effect, $\psi_{ec,N}$ takes account for the group effect when different tension loads are acting on the individual fasteners of a group and $\psi_{M,N}$ considers the effect of a compression force between the fixture and concrete when a bending moment is present.
All these factors are taken equal to 1 because the anchors are not located in the vicinity of an edge, the embedment depth is larger than 100 mm, the anchor plate is loaded centrically and the edge distance is larger than 1,5 $h_{eff}$ thus no bending moments are present.

By substituting all the calculated values in Equation (1), the mean concrete cone capacity of the anchor group is:

$$N_{u,c} = 531,85 \text{ kN} \quad (6)$$

The failure load in the Finite-Element model is

$$N_{u,FE} = 573,14 \text{ kN} \quad (7)$$

4 Conclusion

In this paper, preliminary numerical analyses for circular anchorages are studied. An anchor group of eight headed studs under shear loading is investigated, with three different concrete edge distances and two slab heights. The same group is subjected to tension load, however with one edge distance and slab height. The concrete edge distance influences the load carrying capacity of the anchor group. While increasing this distance from 100 mm to 200 mm seems to increase the failure load, a further increase to 300 mm shows that this influence becomes lower.

The numerical investigations show that when the specimen is loaded in shear, the crack propagation starts from the back row of the anchor groups. Under tension load, the anchor failed by forming a concrete cone.

Future work will involve numerical investigations under inclined loads. Furthermore, another arrangement of the headed studs in the anchor group will be studied.

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CONSIDERATION FOR BENDING IN CASE OF HIGH ATTACHMENTS

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ABSTRACT

In a lot of cases an engineer has to consider bending moments on the fixing elements in the calculation. In the valid standards bending can be neglected if the attachment is made of metal, without intermediate layer or a grout with \( fc \geq 30 \text{N/mm}^2 \) and thickness \( t \leq d/2 \). Further the fixture shall be in contact with the anchor over its entire thickness.

This can’t be fulfilled in many cases. For the fixing of high attachments, e.g. wooden beams, metal hollow tubes, etc. bending concerning the half thickness of the fixing has to be considered.

Also has to be considered if the attachment is full restraint \((\alpha_M=2)\) or if the attachment isn’t restraint \((\alpha_M=1)\). In case of a wooden beam or of a metal hollow tube it can’t be assumed easily if there is the restraint factor on 1, 2 or between 1 and 2. To have the possibility for a proper calculation fischer developed a design method together with IWB and IEA GmbH & Co. KG covering high attachments in different wooden materials and several steel tubes for fixing with the fischer Anchor Bolt FAZ II. The experts report respects also the restraint of the fixing. The result is a consideration above the valid regulations for engineering judgement if necessary.

1 Introduction

The FAZ II anchor bolt is made of galvanised steel, A4 and C to be used in cracked and non-cracked concrete of the strength classes C20/25 to C50/60 included in the European Technical Assessment ETA-05/0069. According to the design provision of EN 1992-4 or EAD 330232-00-0601 for anchors without lever arm must fulfil the following requirements:

- The fixture is made of steel and is in contact with the anchor at least half of the fixture height
- The fixture is fixed directly to the concrete or a suitable intermediate layer as defined in EN 1992-4 or ETAG 001, Annex C.

If these conditions are not fulfilled the shear load shall be assumed to act with a lever arm.

fischerwerke shows that the characteristic and mean failure load remains the same if the fixture is made of timber fixtures with a height \( \leq 300 \text{mm} \). Therefore, shear tests with different timber fixture heights made of spruce and beech were done. The tests include the sizes M10, M12 and M16 of the FAZ II. Based on the tests a Design Model for wooden fixtures was developed. The design model is valid for the fischer FAZ II anchor in combination with the defined wooden parts and steel tubes.
The whole information is given in the “Evaluation Report for fischer FAZ II expansion anchor characteristic bending resistance with wooden and steel tube fixture” developed by fischer and IEA, report N° 13-29, dated on the 28\textsuperscript{th} March 2017. The following essay shows an extract of the Evaluation Report to highlight the developed Design Method for the FAZ II with wooden fixtures.

2 Fixture and attachment

For the fixture steel tubes and timber fixtures where used. For the timber fixture two different kind of timber, beech as hardwood and spruce as softwood where taken to perform the tests. The beech fixture was produced of laminated beech timber. The steel tube was ordered with a thickness of the steel of 12 mm.

2.1 Fixture of timber materials

The timber fixture had a height of 15 mm, 100 mm and 300 mm. On the top a steel plate with a thickness of 5 mm was used to avoid that the washer is pressed into the wooden fixture when the installation torque moment was applied on the anchor. In the reality the FAZ II is available with big washers for wooden construction according to the German standard DIN 1052.

For the tests softwood with a bulk density of 400 kg/m\textsuperscript{3} and hardwood with a bulk density of 720 kg/m\textsuperscript{3} was used. Therefore the results derived are valid for timber with a bulk density larger than the tested bulk density. Alternative for spruce the strength class shall be larger than C35 and for beech the strength class shall be larger than D60 acc. DIN EN 1995-1-1:2010-12.

![Figure 2-1: Examples of wooden fixtures of a) laminated beech and b) spruce.](image)

The loading frame consists of two steel plates connected with two threaded rods. A threaded rod of size M20 to bring up the force was screwed in at the loading side of the frame. The displacement is measured at the opposite end by an LDTV and shows that the difference between the displacement at the anchor and the displacement of the fixture is negligible.

2.2 Fixture of steel material

The fixture made of steel consists of a commercial steel tube with a size 350 mm x 150 mm x tfix with a thickness of the steel of 12 mm and 8 mm respectively. To have the same conditions as for the timber fixture the same steel plate was put on the top of the fixture.
The same loading frame as for the timber fixture with two steel plates connected with two threaded rods was used. The displacement was also measured at the opposite end of the loading frame. Due to the high stiffness of the steel fixture the measurement of the displacement direct at the anchor was omitted.

### 2.3 Test program

The tests were carried out in the laboratory of fischerwerke GmbH &Co. KG and at the laboratory of the Institut for Construction Materials (IWB – FAST3Solution). The test program is shown in the following table:

Table 2-1: Summary of the test program with (green = laboratory of fischerwerke, grey Laboratory of University Stuttgart)

<table>
<thead>
<tr>
<th>Material of the fixture</th>
<th>sprouce</th>
<th>beech</th>
<th>steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>tex, [mm]</td>
<td>15</td>
<td>3x15</td>
<td>100</td>
</tr>
<tr>
<td>dř = 0 mm</td>
<td>-</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>10</td>
<td>-</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>16</td>
<td>-</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>dř = 2 mm</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>2</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>16</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>dř = 4 mm</td>
<td>-</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>10</td>
<td>-</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>16</td>
<td>-</td>
<td>-</td>
<td>3</td>
</tr>
</tbody>
</table>

For all configurations only three tests occurred due to the fact that the scatter is very low. The test program was developed to show the following influences:

- Material of the fixture compared to the standard fixture used in shear tests according to EAD 330232-00-06012.
- Influence of the height of the fixture between 15 mm and 300 mm.
- Influence of the annular gap between \( d_f = 0 \) mm and \( d_f = 4 \) mm.
- Influence to the type of timber (spruce, beech)
In the preliminary tests the fixture with a height of 15 mm failed by splitting. Therefore, no further tests (other sizes or other type of timber) with timber fixture with a height of 15 mm were carried out.

2.4 Concrete member and anchor installation

The tests were performed in low strength concrete C20/25 (non-cracked) and high strength concrete C90/105 (non-cracked). The concrete C20/25 was produced according to the requirements of EAD 330232-00-060. For the performance in the high strength concrete an additional Experts Report (Technical Report assessment of the essential characteristic for fischer FAZ II anchors in concrete strength class C12/15 and C80/95) is available for the FAZ II anchor.

After drilling the holes the drill hole cleaning with a vacuum cleaner was done. The anchors were set by hammer in pre-positioned installation, afterwards the fixture was attached. At least the steel plate was placed on the top of the fixture and fixed by washer and nut. The installation torque moment $T_{\text{inst}}$ was applied with a calibrated torque wrench. After 10 min the torque moment was reduced to 0.5$T_{\text{inst}}$. After the installation the measurement devices were installed and positioned and the tests started.

3 Test Results

3.1 Failure modes

In the tests the observed failure mode was steel failure or failure of the fixture. To avoid concrete failure, the tests at IWB were performed in high strength concrete C90/105.

In case of steel failure the test results can be compared. Almost all anchors with timber fixture failed by steel failure at a large deformation. For the timber fixture with a height of 15 mm the fixture itself failed by a compression failure and splitting of the timber.

![Figure 3-1: Failure mode splitting of spruce with 15mm thickness](image_url)
3.2 Evaluation and assessment of the test results

3.2.1 Load displacement behaviour

The load displacement curves show no plateau at the beginning of the load-displacement curve and a steady increase of the load with increasing displacement. Therefore, all requirements according to ETAG 001, Part 1 and EAD 330232-00-0601 are fulfilled regarding the load displacement behaviour.

3.2.2 Scatter of the load-displacement curves

The evaluation of the scatter of the load displacement curves must be smaller than 40% at a load level of $0.5 \cdot F_u$. The coefficient of variation (COV) was determined for each test series. All tests fulfil the requirements according to ETAG 001, Part 1.

![Figure 3.3 Coefficient of variation (COV) for different fixture materials and fixture heights.](image-url)
3.3 Evaluation of the ultimate shear load $V_u$

3.3.1 Shear loading with steel tube fixture

The evaluation of the ultimate shear load with steel fixtures was evaluated for the tested sizes M10, M12 and M16. The result shows that the mean failure load for a steel tube with $t_{\text{fix}} = 100$ mm and $t_{\text{fix}} = 300$ mm is larger or equal than for the standard configuration according to ETAG 001, Annex A with $t_{\text{fix}} = 30$ mm.

3.3.2 Shear loading with timber fixture

The tests with timber fixtures were performed in the same way as the tests with the steel tube. According to ETAG 001 and EN 1992-4 the configuration with the steel tubes can be interpreted as allowed. The requirements for a steel fixture would be fulfilled. Therefore, the tests with the steel tube have the same fixture height as used for the timber fixture. The results shows the same resistance in case of shear steel failure as observed in the reference test. The evaluation of the mean failure loads shows that the anchorages with the same fixture height are comparable. For a fixture height of $t_{\text{fix}} = 100$ mm the reference tests with a steel tube are always lower than the mean failure load of the anchorages with a wooden fixture. For a fixture height of $t_{\text{fix}} = 300$ mm and the size M10 the mean failure load is slightly lower of a fixture made of for spruce.

![Figure 3-4: Ultimate load $V_u$ for steel tube fixture with a height of $t_{\text{fix}} = 100$ mm and $t_{\text{fix}} = 300$ mm compared with wooden fixtures (spruce and beech).](image)

**Result:** The results show that the tests are comparable and for the FAZ II anchors the mean failure load is not reduced if the fixture is made of timber (spruce or beech) instead of steel.

3.4 Evaluation of the characteristic resistance

3.4.1 Maximum displacement to determine the characteristic resistance

The characteristic resistance of the different investigated anchorages with timber and steel fixtures was evaluated. The evaluation was done in a way that the mean load was determined for a defined displacement. The displacement was evaluated at 1 mm to 5 mm with a step of 1 mm and at 10 mm
displacement that corresponds to a maximum rotation of 6° for a height of a fixture with 100 mm and 2° for a fixture height of 300 mm.

For the anchorages with a wooden fixture (spruce or beech) and with a steel tube the characteristic resistance is evaluated at a displacement fixed to 10 mm. This, due to the fact that the tests with standard fixture show a maximum displacement of 10 mm up to failure level. The tests with standard fixture (t_{fix} = 30 mm) and the steel tube (t_{fix} < 300 mm) shows that the displacement for a steel fixture with t_{fix} = 300 mm is larger than for the standard fixture with t_{fix} = 30 mm. All configurations with a steel fixture are covered by the current approval and the design method according to EN 1992-4 even that the displacement at a level of V_u is quite different.

**Result:** The characteristic resistance of anchors loaded in shear with wooden fixtures (spruce and beech) or steel tube fixtures are evaluated at a displacement of δ=10 mm.

### 3.4.2 Characteristic resistances under shear loading (Maximum)

The maximum characteristic resistance under shear loading is determined in the A5 test series according to ETAG 001. These values are given in the current technical assessment. The values are representing the maximum characteristic shear resistances for steel failure and used as limit if the characteristic resistances evaluated at a displacement of 10 mm would show a higher resistance.

The characteristic resistances for the configuration with wooden fixtures or steel tubes are evaluated based on the mean failure load and carried out that for all tests the characteristic resistance was calculated using the shear stress calculated with the following equation:

\[
\tau_{u,s} = \frac{V_u}{A_s}
\]  

\(V_u\) : Mean failure load observed in the test
\(A_s\) : Cross section area of the threaded part in the assumed shear plane

Using the calculated mean shear stress \(\tau_{u,s}\) and the corresponding standard deviation the characteristic shear stresses for all sizes M10 to M16 can be derived. The calculation was done separately for spruce, beech and steel tube fixtures.

The characteristic resistances are summarised in the following table for displacements \(\delta = 10\) mm. The results are derived from the tests with an increased through-hole clearance. The annular gap \(d_f\) shall be considered separately according to the current approach for standard fixtures. This evaluation is on the safe side.
3.4.3 Restrain factor $\alpha_M$ for bending

To show the difference between a stand-off installation and the shear loading using a non-steel fixture (e.g. spruce or beech) the theoretical restrain factor $\alpha_M$ is calculated.

For a stand-off installation without any fixture in-between the characteristic resistance is calculated using the following equation:

$$V_{Rk,s,M} = \alpha_M \cdot \frac{M_{Rk,s}}{l}$$

(3.2)

$\alpha_M$ = Restrain factor

$M_{Rk,s}$ = characteristic bending resistance given in the ETA

$= 1.5 \cdot W_{el} \cdot f_{yk}$

$l$ = lever arm in the test

$V_{Rk,s}$ = maximum characteristic resistance evaluated from the tests

$\text{Min}(V_{Rk,\delta=10^\circ}; V_{Rk,s})$

If the equation 3.2 is rearranged the restrain factor can be determined:

$$\alpha_M = \frac{V_{Rk,s}}{M_{Rk,s}} \cdot \frac{M_{Rk,s}}{l}$$

(3.3)

$M_{Rk,s}$ = characteristic bending resistance given in the ETA

$= 1.2 \cdot W_{el} \cdot f_{uk}$ assuming a maximal rotation of $10^\circ$

$l$ = lever arm in the test

$V_{Rk,s}$ = maximum characteristic resistance evaluated from the tests

= Load at maximum displacement of 3 mm.

The calculated $\alpha_M$ factor can be used for wooden fixtures or steel fixtures to calculate the characteristic resistance using a lever arm equal to the height of the fixture or defined according to

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<table>
<thead>
<tr>
<th>Material</th>
<th>Size [mm$^2$]</th>
<th>M8</th>
<th>M10</th>
<th>M12</th>
<th>M16</th>
<th>M20</th>
<th>M24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spruce</td>
<td>100</td>
<td>36.6</td>
<td>58</td>
<td>84.3</td>
<td>157</td>
<td>245</td>
<td>353</td>
</tr>
<tr>
<td></td>
<td>300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beech</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>300</td>
<td>Not tested</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>300</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 3-1: Characteristic resistances derived from the performed tests with different fixture materials using the maximum allowed displacement of $\delta = 10$ mm, [kN].
EN 1992-4. The values are derived for a maximum mean displacement of 1 mm and 10 mm. The values are given in the following tables.

<table>
<thead>
<tr>
<th>Material</th>
<th>FAZ II expansion anchor size</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M8</td>
</tr>
<tr>
<td>As [mm²]</td>
<td></td>
</tr>
<tr>
<td>Spruce</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td>36.6</td>
</tr>
<tr>
<td>300</td>
<td></td>
</tr>
<tr>
<td>Beech</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td></td>
</tr>
<tr>
<td>Steel</td>
<td></td>
</tr>
<tr>
<td>100</td>
<td></td>
</tr>
<tr>
<td>300</td>
<td></td>
</tr>
</tbody>
</table>

Table 3-2: Restrain factor $\alpha_M$ for timber fixture and steel fixture for a maximum displacement of 10 mm.

The evaluation shows, that the minimum restrain factor $\alpha_M$ is larger than 10.0 for timber fixtures at the maximum characteristic resistance (displacement $\delta=10$ mm). For the calculation of the maximum characteristic shear resistance of anchors with a timber fixture (beech or spruce) therefore a factor $\alpha_M = 10.0$ is proposed for spruce and $\alpha_M = 15.0$ is proposed for beech.

The evaluation shows, that the minimum restrain factor $\alpha_M$ in case of serviceability (displacement $\delta=10$ mm) is larger than 5.0 for timber fixtures. For the calculation of the maximum characteristic shear resistance of anchors with a timber fixture (beech or spruce) therefore a factor $\alpha_M = 5.5$ is proposed.

4 Design Model

4.1 General proof

The calculation of the characteristic resistance of an anchorage under shear loading with and without edge influence is based on the provisions given in EN 1992-4, Part 4, or ETAG 001, Annex C. For the characteristic resistance of an anchorage with lever arm the following equation applies.

$$V_{Rd,h} \leq V_{Ed} \quad \text{(4.1)}$$

$V_{Rd,h}$ : Design shear resistance for shear stress failure [kN]

$V_{Ed}$ : Design shear loading [kN]

Due to additional bending effects for anchorages with lever arm the following equation shall be fulfilled.
\[ V_{Rd,h,M} = \alpha_M \cdot \frac{M_{Rk,s}}{\gamma_{Ms} \cdot l} \leq \frac{M_{Ed}}{l} \] (4.2)

- \( V_{Rd,h,M} \): Design shear resistance for bending stress failure [kN]
- \( M_{Rk,s} \): Characteristic moment resistance [Nm]
- \( l \): Lever arm [mm]
- \( \alpha_M \): Restraint factor for bending

The material safety factors are given in the ETA or can be taken from the EN 1992-4 if no other information is available.

The smaller of both values \( V_{Rd,h} \) or \( V_{Rd,h,M} \) is decisive for the design.

**Attention:** If a timber fixture is used it must be proofed that the maximum pressures on the timber are not exceeding the given values in EN 1995. The proof can be done in the same way as for bolt connections of timber structures. This is not part of the anchorage proof.

### 4.2 Calculation of the resistance \( V_{Rd,h} \)

#### 4.2.1 Reduction factor for shear stress failure

The design and the calculation of anchors with edge influence and lever arm are based on the assumption, that the load is increased due to the system.

This increased force can be also considered on the resistance by using a reduction factor. This reduction factor \( \alpha_h \) can be determined depending on the system with the following equation:

\[ \alpha_h = \frac{(h_{ef} - a_3)}{(t_{fix} + h_{ef})} \] (4.3)

- \( a_3 \) = Distance of the shear plane to the concrete surface
  - \( = d/2 \), if concrete spalling in front of the anchor is possible
  - \( = 0 \), if concrete spalling in front of the anchor is not possible
- \( t_{fix} \) = height of the wooden fixture
h_{ef} = embedment depth for shear loaded anchors with lever arm ≤ 6d

4.2.2 Characteristic resistance $V_{Rd,h}$ for shear stress failure

When using the reduction factor $\alpha_h$ the characteristic resistance of an anchorage with lever arm can be calculated on the basis of the characteristic resistances without lever arm. The characteristic resistances can be taken from the technical assessment (ETA).

- Steel failure without lever arm $V_{Rk,s}$
- Concrete edge failure without lever arm $V_{Rk,c}$
- Pull-out under shear without lever arm $V_{Rk,p}$
- Pryout-failure without lever arm $V_{Rk,cp}$

The minimum of all design resistances (characteristic resistances divided by the corresponding material safety factor) is decisive:

$$V_{Rd,min} = \min\left(\frac{V_{Rk,s}}{\gamma_{Ms}}; \frac{V_{Rk,c}}{\gamma_{Mc}}; \frac{V_{Rk,p}}{\gamma_{Mc}}; \frac{V_{Rk,cp}}{\gamma_{Mc}}\right)$$

The decisive value $V_{Rd,min}$ of all design resistances must be reduced by the reduction factor $\alpha_h$ in case of an existing lever arm or an wooden fixture.

$$V_{Rd,h} = \alpha_h \cdot V_{Rd,h,min}$$

$\alpha_h$ = Reduction factor due to lever arm
$V_{Rd,min}$ = Decisive design value as the minimum design value of all failure modes

4.3 Calculation of the resistance $V_{Rd,h,M}$

4.3.1 Restrain factor $\alpha_M$

In most cases the current procedure of the design is conservative if bending of the anchors is decisive. For anchors having a non-steel fixture EN 1992-4 the design proof for bending is decisive like for a stand-off installation.

For a stand-off installation $\alpha_M$ depends on the restraint situation of the anchor at the loading point. There are two possibilities for the design:

- $\alpha_M = 1.0$ assuming a free rotation at the loading point
- $\alpha_M = 2.0$ assuming a no rotation at the loading point

The designer can also take a value between $\alpha_M = 1.0$ and $\alpha_M = 2.0$ for a restraint system

If a wooden fixture is used EN 1992-4 applies the same procedure assuming no fixture but a stand-off installation with a lever arm equal to the height of the fixture. This assumption is very conservative due to the fact, that the anchor bolt is continuously supported within the fixture. In this case a factor $\alpha_M > 2.0$ can be applied.
For the anchors FAZ II size M10, M12 and M16 tests were performed and evaluated (see section 3) to determine the restrain factor $\alpha_M$ for wooden fixtures made of spruce and beech. The results shows that at **ultimate limit state** a restrain factor $\alpha_M = 10.0$ for spruce and $\alpha_M = 15.0$ for beech can be applied.

For the **serviceability limit state** the $\alpha_M$ - factor is lower, to consider a maximum mean displacement of 1 mm. For this case the factor $\alpha_M = 5.5$ is applied. To ensure the same displacement behaviour like observed for standard steel fixtures $\alpha_M = 5.5$ shall be taken for the design for the FAZ II anchor. This factor is **product dependent** and only valid for the tested FAZ II anchors in combination with spruce with $\rho > 400 \text{ kg/m}^3$ or beech with $\rho > 700 \text{ kg/m}^3$.

For concrete edge failure only some tests are available. The test shows that up to a fixture height of $t_{fix} = 300 \text{ mm}$ the restraint factor $\alpha_M > 7.0$ can be recommended. This is higher than the applied one.

### 4.3.2 Characteristic resistance $V_{Rd,h,M}$ for bending stress failure

Additional to the resistance in case of shear stress failure $V_{Rd,h}$ the design resistance in case of bending stress failure $V_{Rd,h,M}$ shall be calculated. Using this equation ensures also that the serviceability limited state.

$$V_{Rd,h,M} = \alpha_M \cdot \frac{M_{Rk,s}}{t_{fix} + a_3} \cdot \frac{1}{\gamma_{Ms}}$$

(4.6)

- $a_3$ = Distance of the shear plane to the concrete surface
  - $= d/2$, if concrete spalling in front of the anchor is possible
  - $= 0$, if concrete spalling in front of the anchor is not possible
- $t_{fix}$ = height of the wooden fixture or steel tube
- $\alpha_M$ = Restrain factor derived from tests

### 4.4 Characteristic resistances for FAZ II with wooden fixture

The characteristic resistances for the anchors FAZ II size M10, M12 and M16 with wooden fixture can be determined with the following equations. The method is simplified in a way that $a_3$ is assumed to be 0 mm. For this case $\alpha_M$ is taken as 5.0 and not 5.5. The minimum of the following characteristic shear resistances are decisive:

$$V_{Rk,s,h} = \left(\frac{h_{ef}}{h_{ef} + t_{fix}}\right) \cdot V_{Rk,s}$$

(4.7)

$$V_{Rk,c,h} = \left(\frac{h_{ef}}{h_{ef} + t_{fix}}\right) \cdot V_{Rk,c}$$

(4.8)

$$V_{Rk,cp,h} = \left(\frac{h_{ef}}{h_{ef} + t_{fix}}\right) \cdot V_{Rk,cp}$$

(4.9)

$$V_{Rk,p,h} = \left(\frac{h_{ef}}{h_{ef} + t_{fix}}\right) \cdot V_{Rk,p}$$

(4.10)

$$V_{Rk,h,M} = \alpha_M \cdot \frac{M_{Rk,s}}{t_{fix}}$$

(4.11)

Using the following values:

- $h_{ef}$ = effective embedment depth $\leq 6d$
- $t_{fix}$ = height of the fixture $\leq 300 \text{ mm}$
Characteristic resistance in case of steel shear failure without lever arm
\[ V_{Rk,s} \]

Characteristic resistance in case of concrete edge failure without lever arm
\[ V_{Rk,c} = k_8 \cdot N_{Rk,c} \]

Characteristic resistance in case of pryout failure without lever arm
\[ V_{Rk,p} = k_8 \cdot N_{Rk,p} \]

\[ \alpha_M \] Restrain factor derived from tests
\[ = 5.0 \text{ for wooden (spruce and beech) fixtures and FAZ II.} \]

## Summary

The development of the new design model shows the practicability for the expansion anchor FAZ II used in combination with wooden fixtures with a height \( t_{fix} \leq 300 \text{ mm} \). The tests show that the mean ultimate load and the characteristic resistance of the anchors FAZ II are not negatively influenced if the fixture is made of timber instead of steel.

The results show that the shear displacement is increased if the fixture is made of timber. For comparison the displacements observed for a standard fixture according to EAD were evaluated and the tests with a wooden fixture are assessed for the same displacement. The assessment shows, if a restrain factor of \( \alpha_M = 5.0 \) is used (continuous support in the fixture) the serviceability level is the same.

To account for the lever arm the characteristic resistances shall be reduced with a factor \( \alpha_h \) to account for the lever arm. This factor depends on the geometric system and therefore on the embedment depth and fixture height.

## References:


2. European Technical Assesment, ETA-05/0069, fischer Ankerbolzen FAZ II Torque controlled expansion anchor of sizes M8, M10, M12, M16, M20 and M24 for use in cracked and uncracked concrete, Deutsches Institut für Bautechnik.


6. European Assessment Document - EAD 330232-00-0601, MECHANICAL FASTENERS FOR USE IN CONCRETE, October 2016, by EOTA.
Johannes Braun

INFLUENCE OF SPECIMEN GEOMETRY ON ANCHOR BEHAVIOR IN SEISMIC CRACK MOVEMENT TESTS

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ABSTRACT

It is generally acknowledged that one of the most critical test to capture the behavior of a post-installed fastener under seismic action is the so-called “seismic crack movement test”, which consists in applying a constant load to a single fastener installed in a crack subjected to opening and full closing cycles. The paper presents numerical simulations and experimental results of crack movement tests on large size post-installed anchors which show a strong influence of the geometry of the concrete specimen in which the anchor is installed. As first assessment, the intrinsic limit of applicability of a concrete specimen unable to allow a full theoretical development of a concrete cone is put in evidence. By comparing results of seismic crack movement tests using two different specimens, it is noticed how splitting force generated by the anchor affects the restoring of zero crack opening when increasing the number of cycles. Increasing the size of concrete element has a beneficial effect: the drift between crack opening measured at the top of concrete element and at the embedment depth is limited, limiting the effects of bending induced in the concrete specimen. Such difference also affects the anchor’s performance in residual pull-out test, where the possibility of developing a full concrete cone needs to be preserved while avoiding strut and tie mechanisms. To improve the regularity of the crack plane, a feedback control using the crack opening measure is used and discussed. The results are compared with systems using an open loop control.

1 Introduction

The crack movement test is one of the most demanding test in seismic assessment of post-installed fasteners. It can be stated that it is generally more decisive than cyclic loads tests in determining final anchor performances.

Reviewing the theoretical motivations for crack protocols, one may state that kinematic of crack must be independent from anchor behavior; hence two questions may arise: 1) which kind of regularity is requested to crack opening during cycles and how to measure it. 2) which kind of influence the variability of test conditions (i.e. steel reinforcement ratio, bond parameters, concrete geometry) may play in determining anchor’s performance.

In this paper, displacement of anchor during crack cycles and residual pullout capacity are supposed to be relevant in evaluating the dependence and results for the two different systems that are presented. The most important effect of geometry dependence is in crack closure; the issue is here
approached analytically and numerically, showing what are the variables need to be harmonized in future standard approach.

2 Crack protocols

In the pioneering work of Wood, Hutchinson the origin of crack protocols for anchored system is explained. The non-linear behavior of five concrete prototype frame and two coupled wall-frame designed according ACI 318 and ASCE 7 was investigated. Different number of storeys and building typologies produce the dynamic variability intended to represent medium rise building subjected to high seismic hazard. As a result 2346 time histories of curvature (converted in crack opening via Gergely, Lutz formula) treated with rain-flow counting were assumed as statistical base to define a crack protocol. In Figure 1 results are shown: non-dimensional crack opening \( w/w_y \) was binned in 10 representative value from 0.1 to 1.0. Constant amplitude cycles of crack opening are supposed to replicate in damage one of the 2346 time histories according the Palmgren, Miner rule. In order to obtain crack protocols the value of the non-dimensional crack opening should be multiplied the values by \( w_{max} = 0.5 \) mm or \( w_{max} = 0.8 \) mm. In this case they’re treated as extreme values with probability of exceedance related to the limit state: serviceability for 0.5 mm, suitability for 0.8 mm. In ETAG 001 – Annex E a unified crack protocol (discussed mainly by Mahrenoltz) was assumed: 45 cycles increasing from 0.1 mm to 0.5 mm are performed with \( N_{w1} \) axial load imposed to the anchor; than the load switch to \( N_{w2} \) and crack opening is cycled 14 times with increasing step amplitude form 0.6 mm to 0.8 mm. \( N_{w1} \) and \( N_{w2} \) are \( 0.4 \cdot N_{ccr} \) and \( 0.5 \cdot N_{ccr} \) respectively, being \( N_{ccr} \) the mean of reference tests for concrete break-out in static crack opening.

![Figure 1: Wood, Hutchinson results for crack protocols: (a) rain-flow count results for different building prototype; (b) results after statistical analysis](image-url)
3 Test evidence

3.1 Test setup

In Figure 2 two systems are presented: (a) system of 1000 kN capacity for the horizontal cylinder fixed to the strong floor; (b) system of 2000 kN capacity self-equilibrated. In Table 1 specifications for concrete members are given.

![Figure 2 test setup for crack movement test: (a) 1000 kN capacity; (b) 2000 kN capacity](image)

<table>
<thead>
<tr>
<th>I (mm)</th>
<th>b (mm)</th>
<th>h (mm)</th>
<th>bars</th>
<th>Diameter (mm)</th>
<th>$L_{\text{deb}}^{(1)}$ (mm)</th>
<th>$L_{\text{transf}}^{(2)}$ (mm)</th>
<th>$\rho^{(3)}$ (%)</th>
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<tr>
<td>Concrete member 340 x 500</td>
<td>1480</td>
<td>500</td>
<td>340</td>
<td>4</td>
<td>16 $\div$ 20</td>
<td>-</td>
<td>740</td>
</tr>
<tr>
<td>Concrete member 700 x 500</td>
<td>1480</td>
<td>700</td>
<td>500</td>
<td>8</td>
<td>16 $\div$ 20</td>
<td>40</td>
<td>700</td>
</tr>
</tbody>
</table>

(1) Debonding length
(2) Transfer length in bond diagram according Leonhardt
(3) Steel ratio ($A_s / A_{c, \text{gross}}$) being $A_{c, \text{gross}} = b \cdot h$
Crack is measured at the top of concrete specimen and at the embedment depth of the installed anchor. Two points of sampling per side are monitored via LVDT transducers (0.001 mm precision), the mean value between side is taken as reference variables in tests.

Actuator position in case of system (a) in Figure 1 is based on open loop control: crack is only measured and the motion is inverted if target of crack opening is achieved during cycles. Closed loop control, instead, is implemented in system (b): the relative position of actuators depends upon the difference between the crack opening values measured at two side of concrete specimen, which is assumed as control variable. Closed loop is maintained in unloading until crack closure is reached, then actuators are forced in same position to apply compressive force without eccentricities (see Figure 3). Crack opening difference used as feedback signal in controlling the actuator position guarantees a kinematic constraint to the crack plane in favor of repeatability.

![Figure 3 Example of closed loop control](image)

### 3.2 Force distribution

The effect of tensile force application for the anchor is a parasitical flexural behavior for the concrete member which provokes a gradient in measuring the crack opening along the depth (see Figure 4). The effect increases by increasing the load, which basically corresponds to the increase in diameter in case of post-installed anchors. Increasing the tensile load may also cause an increase in splitting force generated by the anchor in the crack plane which is resisted by the reinforcing bars only. Furthermore, application of tensile force in concrete is guaranteed by the bond-slip mechanism, however unloading path makes one able to recover steel elastic deformation only, slip is recovered by applying a compressive force. The last is taken up by steel and concrete at the level of cross section. The amount of stress that concrete is able to transfer in bearing zone during compression depends on the degree of crack closure, it will be shown in the following paragraphs that this mechanism is strongly influenced by boundary conditions of test setup and in particular those characterizing bond of rebars.
In Figure 5 diagrams of crack opening and actuator load (negative load indicates compression of the concrete member) for seismic crack movement according to ETAG 001\textsuperscript{2} are presented. Large diameter adhesive anchor, M30 with 210 mm of embedment depth, was tested in both concrete specimens previously discussed. Bending behavior was prevented by confinement.

Diagram can be divided in the following paths, e.g. in Figure 5 (a): 1) tensile path at crack opening value; 2) unloading path up to zero actuator’s load; 3) slip recover with compressive force; 4) compressive force applied to concrete with crack closure. Tensile load is exerted without change in stiffness, meaning that tension stiffening is not relevant at steady state of cycles. In 2) steel deformation of rebars is recovered while slip recovery happens along path 3). Path 4) can be recognized in an abrupt change in stiffness. Negative value of crack opening is an effect of LVDT’s measure length, even though no specific theoretical meaning is associated to that. Residual crack at zero actuator load is evident in case (a), where splitting force is reacted by 4 bars only. The effect is mitigated by increasing the number of bars in the cross section, as in case (b). Splitting increases evidently at last 15 cycles, where axial load for anchor passed from $N_{w1}$ to $N_{w2}$ with a 20% increasing (e.g. $N_{w1} = 80$ kN and $N_{w2} = 96$ kN). It must also be remarked that those stages of crack opening introduce the higher slip in reinforcement bars during path 1, hence path 3) (where slip recover takes place) is increased as well. As a matter of fact, in case (a), even at the max compression, path 4) was not observed for the very last cycles and crack was not properly closed.

Figure 5 Crack opening - Actuator load law (a) test in 1000 kN system; (b) test in 2000 kN system
Crack opening at zero actuator load seems in contradiction with the theory of crack protocols prescribing that it should be obtained stepwise without any influence between two consecutive steps. This requires to maintain a perfect elastic behavior crack opening and actuator load diagram as proved by Hoehler. Moreover, difference between two test setups in developing paths 3) and 4) affects repeatability, in fact compressive force transferred to the anchor may change at crack closure. In Figure 6 crack opening values at zero actuator load are shown for post - installed expansion anchors tested in system (a) of Figure 2. Crack opening \( w_0 \) is measured after cycle 45 (last cycle for \( w = 0.5 \) mm) performing crack movement seismic test according ETAG 001.2.

Applied load \( N_w \) varies from 5 to 36 kN, increasing the diameter. Displacements at Damage Limitation State (DLS) are reported in Figure 7 (a) for large diameters anchor (i.e M16, M20, M24), with 7 mm threshold as requirement for assessment at DLS.

For sleeve type anchors, increasing the value of \( w_0 \) corresponds to higher displacements either for DLS and Ultimate Limit State (ULS). In Figure 7 (b) the ratio \( \delta_{DLS} / \delta_{ULS} \) is plotted as a function of \( w_0 \). Anchor’s behavior is affected by the residual crack opening mostly during last cycles (i.e. from DLS to ULS, \( w = 0.5 \) mm to \( w = 0.8 \) mm) where the most demanding slip of rebars is involved. For wedge type anchors the trend is not so clear, since \( w_0 \) exceeds 0.125 mm in two cases only, considering it as a limit value.

Here an example of analytical evaluation of \( w_0 \) is proposed:

\[
 w_0 = w_{\text{slip}} - \varepsilon_s \cdot L_{db} \tag{1}
\]

Where \( w_{\text{slip}} \) is crack opening value due to slip of rebars; \( \varepsilon_s \) the deformation of reinforcing bars recovered at the end of the unloading path. In this case slip is assumed completely unrecovered at the end of the unloading path, as experienced by Balazs. The term \( w_{\text{slip}} \) in the equation is quite difficult to evaluate since it depends on the state of stress of the reinforcement bars, i.e. \( w_{\text{slip}} (\sigma_s) \). \( \sigma_s \) can be roughly estimated as the superposition of three terms during the tensile loading path:

\[
 \sigma_s = \sigma_F + \sigma_{\text{spl}} + \sigma_{\text{flex}} \tag{2}
\]

Where \( \sigma_F \) is the stress induced by pulling the rebars to acquire a target value of crack opening (w) \( \sigma_{\text{spl}} \) is the contribution due to splitting of the anchor in the crack plane evaluated as

\[
 \sigma_{\text{spl}} = (F_{\text{spl}} / n_s \cdot A_s) \tag{3}
\]

\( F_{\text{spl}} \) the splitting force induced by \( N_w \); \( A_s \) and \( n_s \) the area and number of reinforcing bars reacting to splitting force;

\[
 \sigma_{\text{flex}} = \left[ (N_w \cdot L_{\text{frame}} / 4) \cdot d_s / (E_s \cdot J_s) \right] \cdot L_{db} \tag{4}
\]

is the contribution due to flexural behavior being: \( L_{\text{frame}} \) the span of the reacting frame; \( d_s \) distance between reinforcing bars in cross - section; \( J_s = 2A_s \cdot (d_s/2)^2 \) the inertia momentum in cracked section considering bars only (placed symmetrically with respect the centroid). As a matter of fact, the amount of slip (fixing the target value of crack opening) depends on the state of stress induced by the anchors in the crack plane, this parasitical effect is reacted clearly with dependence on concrete specimen.
Figure 6 Residual crack opening at 45 cycle $w_0$: (a) top measure $w_{0\text{top}}$; (b) difference top – $h_{\text{eff}}$

Figure 7: (a) Anchor’s displacement after cycle 45 ($\delta_{\text{DLS}}$); (b) displacement ratio $\delta_{\text{DLS}} / \delta_{\text{ULS}}$

3.3 Modes of failure

For large diameters anchors, capability to develop concrete cone is one of the requirement for concrete specimen geometry. Eligehausen et al.\textsuperscript{7} and Marhenoltz\textsuperscript{8} noted that the requirement for depth of the test member is $>2 \cdot h_{\text{eff}}$ (see Figure 8 (b) for reference). Lack of depth may lead to secondary mode of failure as splitting of reinforcement observed in Figure 9 (a), tested anchor was M24 expansion sleeve with 150 mm of embedment depth.

During pullout tests (see Figure 8 (a) and (c)), crack opening evolves according the bending behavior: increasing the sectional inertia in this case has strong beneficial effect, in Figure 8 (c) crack opening difference (average top measure – average $h_{\text{eff}}$ measure) is shown for reference seismic tests, i.e. pullout test with 0.8 mm static crack opening\textsuperscript{2}. Pullout capacities seem to be not affected by crack gradient, instead peak displacement differs since a V shape of the crack along the depth favors the expansion cone’s slip.
Figure 8 Pullout tests with 0.8 crack opening: (a) load-displacement curves; (b) modes of failures; (c) crack opening difference; (d) crack opening with gradient along depth

Figure 9 Photograph of modes of failure: (a) splitting of rebars in 340 x 500 concrete element; (b) semi–cones failure in 700 x 500 concrete element
4 Analytical and numerical model

The evolution of compressive in concrete stress during crack closure is investigated.

4.1 A simple model formulation

A simple one dimensional model is presented. The scope is to catch the behavior of concrete and steel during cycles of crack opening and crack closure, the model does not consider the presence of anchor in the crack plane. See Figure 10 for details.

The model is based on the assumption that response is different in tension and in compression (as remarked in Figure 5).

![Figure 10: 1D model for crack movement](image-url)
The response of the element for the stress history shown in Figure 10 is here developed with the following hypothesis: a) undeformed reference configuration (i.e. $\varepsilon^0 = 0$); b) stress $\Delta\sigma^{0-1}$ is such that the slider element is active being $w_{sl}$ the amount of slip; stress $\Delta\sigma^{2-3}$ is such that the slip $w_{sl}$ is not recovered.

In path 0–1 the deformation of the steel + slip series of elements (subscript “s+s” in equations) can be written as follow:

$$\Delta\varepsilon^{0-1}_{s+s} = w_{sl}/l_{transf} + \Delta\sigma^{0-1}/E_s$$  (5)

At the end of step 1 the total deformation is

$$\varepsilon^1_{s+s} = \varepsilon^0 + \Delta\varepsilon^{0-1}_{s+s} = w_{sl}/l_{transf} + \Delta\sigma^{0-1}/E_s$$  (6)

In the unloading path the deformation of the steel is recovered while the slip is fixed, easily at the end of step 2 the total deformation is

$$\varepsilon^2_{s+s} = \varepsilon^1 + \Delta\varepsilon^{1-2}_{s+s} = w_{sl}/l_{transf} + \Delta\sigma^{0-1}/E_s - \Delta\sigma^{0-1}/E_s = w_{sl}/l_{transf}$$  (7)

In compression ($\Delta\sigma^{2-3}$) concrete is active and deforms of $-\Delta\varepsilon^{2-3}_c$, steel deforms with $-\Delta\varepsilon^{2-3}_s$. Total deformation at the end of step 3 can be written as follow:

$$\varepsilon^3_{s+s} = \varepsilon^2 + \Delta\varepsilon^{2-3}_{s+s} = w_{sl}/l_{transf} - \Delta\sigma^{2-3}_c/E_c$$  (8)

for the steel + slip series of element, and for concrete

$$\varepsilon^3_c = \varepsilon^3_{s+s}$$  (9)

Since the steel + slip element and concrete act in parallel, compatibility holds. It is possible to express the $\Delta\sigma_c$ as a function of $\Delta\sigma_s$ (superscript were eliminated for sake of synthesis).

$$\varepsilon^3_c = \varepsilon^3_{s+s} \rightarrow \Delta\sigma_c = \Delta\sigma_s - w_{sl}/l_{transf} \cdot E_c$$  (10)

Where $n = E_s / E_c$

Assuming such kinematic behavior, equilibrium in case of a reinforced concrete element under compression force $N$ is

$$N = \Delta\sigma_c \cdot A_c + \Delta\sigma_s \cdot A_s$$  (11)

Developing (10) and (11) it is possible to show how the amount of compression stress in concrete varies depending on steel ratio ($\rho$), slip ($w_{sl}$), transfer length ($l_{transf}$)

$$\Delta\sigma_c/\sigma_{ref} = 1/(1+n\cdot\rho) - w_{sl}/l_{transf} \cdot E_c \cdot [(n\cdot\rho)/(1+n\cdot\rho)] \cdot 1/\sigma_{ref}$$  (12)

In (12) $\sigma_{ref}$ refers to mean stress which has to be accounted for crack closure. ETAG 001$^2$ prescribes one of the two values 0.1$\cdot f_{cc}$ or 0.15$\cdot f_{cc}$, the last is assumed if crack closure was not below 0.1 mm. The ratio between the expected value and the actual one is taken as measure of redistribution.

Results are shown in Figure 11, as expected compressive stress in concrete diminishes as the value of steel ratio $\rho$ increases. Several parameters were involved in calculation, here the choice was to show the dependence on transfer length and debonding length. Those parameters vary quite a lot looking at
concrete specimens typically used in crack movement tests by different testing labs, as for the ones reported by CSTB\(^9\). The other parameters were selected according to common testing experience.

Figure 11 Reduction in concrete compression stress: (a) transfer length dependence; (b) debonding length dependence

4.2 Numerical analysis

Simulation of crack movements was performed using software ATENA. Concrete element, modeled in plane stress condition, was subjected to crack opening by pulling rebars and compression force applied at the end section, loads were assumed quasi–static. Problem geometry and material data are illustrated in Figure 12 (a). Results confirm reduction in compressive stress for concrete during crack closure, the amount of reduction is nearly 25% comparing the first and last cycle. As mentioned before such behavior is mainly due to slip of reinforcement bars which increases according the assumed crack history.

Figure 12: (a) model overview; (b) results for crack opening and compressive stress in concrete
It is here remarked that the model does not account for fatigue in bond since the number of cycles is not so high; however, increase in slip value should be expected in case of hundreds of load cycles were involved. As it happens in most of the cases, the concrete test member has more than one induced crack, and multiple crack movement tests are done consecutively, hence number of slip cycles can be relevant. The prediction of cycle dependent is herein reported. Slip \( s_n \) as a function of the initial slip \( s_0 \) and the number of cycles \( n \) can be written:

\[
s_n = s_0 \cdot (1+k_n) \quad \text{with} \quad k_n = (1+n)^{0.107} - 1
\]

(13)

5 Conclusion

Influence of geometry of concrete test member in seismic crack movement tests is characterized by the following aspects:

1. Parasitical actions of bending and splitting force induced by applying a tensile force on the anchor are reacted by reinforcing bars only at crack plane. Hence an increase in the amount of reinforcement favors to prevent drift in residual crack opening at zero actuator load.
2. Slip of the reinforcement bars during crack opening is responsible for redistribution of the compressive force. The amount of slip depends on type of rebars, transfer length and debonding length. Such effect is general more evident in last cycles for a seismic crack movement tests, when values of 0.5 mm up to 0.8 mm of crack opening are reached.
3. Compressive stress in bearing zone of anchors strongly depends on the steel ratio and on other parameters characterizing bond behavior of reinforcement bars. Compressive force for the concrete test member should be increased for the very last cycles (performing an ascending crack movement protocol) to gain the lost stress. Steel ratio should be limited to \( 0.50 \div 0.75\% \) as a compromise to fulfill also the previous requirements.

6 Acknowledgement

The work presented in this paper was supported by Material Testing Laboratory (LMP) of Politecnico di Milano. Special thanks are due to Daniele Spinelli for his assistance.

References

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DEVELOPMENT OF SLAB-TO-WALL CONNECTIONS ASSISTED BY EXPERIMENTS AND NUMERICAL SIMULATIONS

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ABSTRACT

Slab to bearing wall connections are used to join precast or cast-in-place concrete floor or roof members to bearing walls. They transfer a vertical load from the horizontal system and, sometimes, diaphragm action by providing moment and shear resistance. The tested connection is part of a novel construction technology requiring a fast and safe fabrication of buildings. In contrast to traditional solutions, where moments in frame corners are transferred by appropriate detailing of bar reinforcement, tensile forces are resisted by inserts anchored in concrete.

Structural functionality of inserts was verified by the experimental investigation of a slab-to-wall joint subjected to moment loading. A full-scale specimen was tested whereas the force, deformation and strain responses were monitored. The experiment was simulated numerically by program ATENA. It helped to analyze in detail the structural behavior observed experimentally and explain the resistance model exploiting inserts. The experimental and numerical results were analyzed with emphasis on the anchoring capacity of inserts.

The project demonstrates the efficiency of development process based on combining laboratory testing with numerical simulation.

1 Introduction

Slip-formed construction of major concrete building cores can be enhanced by the provision of connections that allow the reinforcing bars in floor slab to be mechanically connected to inserts in the wall.

A state-of-the-art design procedure is offered in the fib document Design of Anchorages in Concrete\textsuperscript{1}, where a procedure to determine tension loads on a group of anchors due to applied bending moment is proposed. Elastic analysis is recommended for the brittle failure mode of anchorages under eccentric tension load, which assumes a linear distribution of strain between the anchorages and a linear relationship between the stress and strain. The tensile forces in anchorages are calculated by applying the method of reinforced concrete section, shown in Figure 1. The background research is treated in detail in references\textsuperscript{2,3}. 

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Concrete cone resistance of an anchor group, subject to a bending moment, can be calculated using Eq.(1):

$$N_{R,c}^0 = N_{R,c}^0 \cdot \psi_{A,N} \cdot \psi_{s,N} \cdot \psi_{ec,N} \cdot \psi_{re,N}$$  (1)

Where:

- $N_{R,c}^0$ = Resistance of a single anchor without edge and spacing effect,
- $\psi_{A,N} = \text{factor accounting for the geometric effects of spacing and edge distance},$
- $\psi_{s,N} = \text{factor accounting for the influence of edges of the concrete member on the distribution of stresses in the concrete},$
- $\psi_{ec,N} = \text{factor accounting for the group effect when different tension loads are imposed to the individual anchors},$
- $\psi_{re,N} = \text{factor accounting for the negative effect of closely spaced reinforcement in concrete member on the strength of anchors with embedment depth } h_{ef} < 100\text{mm (not relevant to this study as } h_{ef} > 100\text{mm, } \psi_{re,N} = 1).$

The quantities in Eq.(1) are explained below:

$$N_{R,c}^0 = k \cdot \sqrt{f_{cc200}} \cdot h_{ef}^{1.5}$$  (2)

Where:

- $k = 15.5$ for calculating mean concrete cone strength of anchors, Eligahausen,$^2$,$^3$
- $f_{cc200} = \text{concrete compressive strength from testing of 200mm cube specimen},$
- $h_{ef} = \text{effective embedment depth of anchor},$
- $\psi_{A,N} = A_{c,N} / A_{c,N}^0$  (3)

Where:

- $A_{c,N} = \text{Actual projected area of concrete cone of the anchorages at the concrete surface}$
- $A_{c,N}^0 = 9h_{ef}^2$ which is the reference area of the concrete cone of an individual anchor projected on the concrete surface
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\[ \psi_{s,N} = 0.7 + \frac{0.3c}{1.5h_{ef}} \leq 1.0 \]  
(4)

Where:  
\( c = \) Smallest edge distance in the anchor group

\[ \psi_{ec,N} = \frac{1}{1 + 2\varepsilon_{N1}/3h_{ef}} \leq 1.0 \]  
(5)

Where:  
\( \varepsilon_{N1} = \) Eccentricity of the resulting tensile force acting on the tensioned anchors with respect to the centre of the tensioned anchors

Design according to the above procedure was verified by a full-scale testing and numerical simulation. The test specimen comprised of a cantilevered floor connected to a restrained wall, where the floor is loaded to generate a bending moment to act on the cast-in Insert assemblies. The bending moment, shear force, strains in the starter bars and floor and wall deflections at various positions were determined for all stages of loading. The experimental investigation was supported by a numerical simulation and served in the stage of the test planning for a prediction of experimental behavior, and later, in the stage of evaluation of test results.

2 Test Specification

2.1 Connections

The cast-in insert assembly consisted of insert, Figure 2, ensuring the anchorage in the wall and starting bars ensuring the anchorage in the slab. The position of inserts in the wall before casting of concrete is shown in Figure 3. The cross section of the test specimen is shown in Figure 4. The floor slab with depth of 300mm is provided with a double layer of reinforcement, and two rows of four starter bars 600mm length at 70mm cover top and bottom. The two cast-in insert assemblies were used to connect the wall to the floor, where the eight starter bars were mechanically screwed into the insert by no less than five rotations. Once the starter bar lengths were connected to the cast-in Insert the floor was cast in-situ to complete the assembly.

Figure 2: Cast-in Insert Geometry.
2.2 Construction

The test specimens were constructed in two stages. The first stage being forming, reinforcing and casting the wall units, including the Reidbar assemblies. The second stage was carried out after stripping the wall units, and placing them vertically in an as-tested orientation, shown in Figure 5. Then the floor units were formed, reinforcing placed, including the eight mechanically connected starter bars, and then concrete poured. The specimen was constructed together, ensuring that the floor-to-wall connection remained intact and unaffected by specimen placement and movement.

Prior to the assembly of the reinforcing of the floor, strain gauges were fitted to three of the starter bars, Figure 6.

3 Test setup

The test setup is shown in Figure 5. The wall section was restrained uniformly along its width to the strong floor at the distance of 400mm from the floor. The load was applied uniformly along the width of the floor by reacting against the strong wall at the distance of 1700mm from the face of the wall. A hydraulic cylinder and a 250kN loadcell were used to apply and measure the load during the test. Five Linear Variable Differential Transformers (LVDTs) were used to measure the displacement at various positions. Two of LVDTs were placed at the distance of 1500mm from face of the wall on...
the left and right sides of the specimen to measure the floor displacement and the average of the two displacements was used for the load-displacement response. In order to check the support constraints (a possible support movement) one LVDT was used to measure the displacement of specimen at 750mm above the face of wall and two LVDTs were used to monitor the movements in the wall. The load cell, LVDTs and strain gauges were connected to a datalogger to record the measurements during test.

Figure 5: Experimental setup

Figure 6: Strain gauges on starter bars.

Figure 7 – View of LVDT positions.
4 Finite Element Model

A finite element model was developed reflecting details of the test specimen as shown in Figure 8. The numerical simulation of tests was made by ATENA software. A finite element type chosen for the wall is the isoparametric low order tetrahedron with one integration point and for the slab and inserts the hexahedron with eight integration points. A 1D truss element embedded in concrete elements, with axial resistance only, is used for the reinforcement bars. Interface elements were applied for surfaces between slab and wall and on the insert surface to concrete. A symmetry of the specimen along the vertical plane is used to reduce the model size.

A material model for concrete is the fracture-plastic model, see Cervenka. In this model a brittle behavior characteristic to a pull-out mode of failure relevant to the investigated problem is described by fracture mechanics, which is implemented within a smeared crack concept with a crack band method.

The concrete strength of the test specimen was 45 MPa as determined in the laboratory tests. Other material parameters for concrete were estimated based on the known code formulae as follows: Elastic modulus 34000 MPa, tensile strength 3 MPa, fracture energy 50 N/m. The yield strength of reinforcing bars in the wall as well as in the slab was 530MPa and the one for the anchoring bars R20 rigidly connected to the inserts was 571 MPa. Loading and support plates were considered elastic. The loading applied near the slab end, was accomplished by the prescribed displacement \( d \) in increments of 0.5mm. The loading force \( P \) was calculated as a reaction for the given displacement. The solution required about 90 steps to the failure.

![Overall view](Overall view)  
Reinforcing and anchoring inserts

Figure 8: Numerical model. Loading and supports shown schematically.

5 Results

In the experimental investigation three nominally identical floor-to-wall joint specimens were tested. The overall performance of the specimens in the test is documented by the load-displacement
diagrams shown in Figure 9. The diagram compares the response curves of experiments and simulations.

The prediction provided by concrete cone capacity method according the fib Bulletin 58\(^1\) (which gives the ultimate load only) is 45.5kN. The prediction made by a numerical simulation (green curve) was made in the stage of test planning on an originally designed test specimen. The real tests had slightly changed position of the loading force and simulated response (marked by red curve) was made on the actual geometry (the loading force was acting at a distance of 1700mm to the wall). Furthermore, in this simulation the interfaces on insert surfaces were included in the model (which was not the case in the stage of test prediction). The maximal loads obtained from tests and simulations are summarized in Table 1.

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<td>(P_u) [kN]</td>
<td>45.5</td>
<td>83.1</td>
<td>79.4</td>
<td>68.4</td>
<td>92.2</td>
<td>80.0</td>
<td>86.4</td>
</tr>
</tbody>
</table>

The resistance according to fib Bulletin 58\(^1\) is conservatively lower than the observed ultimate load in the tests, which can be explained by the beneficial effect of the compression stress, due to the applied bending moment, near the concrete cone. The compressive stress in vicinity of the concrete cone has a confinement effect which increases the resistance of the concrete cone failure in tension.

The main objective of the investigation, which was the functionality of inserts connected to the bars R20 anchored in the slab, is illustrated in Figure 10 to Figure 13. The experiments provided data about cracks on the concrete surface and strains in the anchoring bar R20. The point of strain location is marked as SG9 in Figure 6. It is the bar on the tension side near the wall center.

The simulation provided extensive data on stress and strain states and damage due to cracks: The crack pattern inside of the wall depicting the failure cone, Figure 10, stress in concrete near the inserts, Figure 13. They describe the anchoring action of inserts. The stress evolution in the Reid starting bars as obtained by simulation is shown in Figure 12. The process of failure was dominated by a pull-out behavior of inserts in the wall, which was observed in both, experiment and simulation.

The mode of failure was not due to a yielding of reinforcement R20. The elastic strain limit of steel is 0.00285 while the yield stress was 571 MPa. In the experiments, only one strain gauge out of nine (SG9 in test 3) indicated a steel yielding. In the rest of measured points the strain was below the elastic strain limit. Only the failure mode of test 3 with the greatest maximal force could be considered as a mixed steel and concrete failure. In the simulations, the maximal stress and strain was 498 MPa and 0.0024 respectively, thus within the elastic range.

Following the comparison of experimental and simulation data it was concluded that the numerical model describes well the average overall behavior of test specimens. However, some significant differences were found in the individual tests as can be observed in Figure 9. This can be explained by the brittle type of failure in the insert vicinity, which is due to a random effect of concrete non-
homogeneity. Even greater differences were observed in strains of anchoring bars between individual tests and between the test and the simulation. A source of these discrepancies may be in the mechanical connection of bar R20 to inserts. This is a motivation for a future development.

Figure 9: Load displacement diagrams of tests and simulations.

![Figure 9: Load displacement diagrams of tests and simulations.](image)

Figure 10: Crack patterns in experiment and simulation.

![Figure 10: Crack patterns in experiment and simulation.](image)
Figure 11: Load-strain relation in R20 bar at SG9 in tests and simulation.

Figure 12: Stress evolution in the anchoring bars R20 at peak load in the simulation.
Development of floor-to-wall connection using headed anchors installed in the walls was based on experiments and supported by numerical simulations. The failure mode was brittle due to a pull-out resistance of the inserts in the concrete.

The resistance based on the concrete cone capacity method (CCD) as reflected in the fib Bulletin 58 offers a safe design, which was confirmed by test and numerical simulations. The low value of predicted strength as compared to the observed failure load is attributed to the beneficial effect of the compression stress zone adjacent to the concrete cone, which has not been considered in the CCD method.

The numerical simulation with ATENA software reflected well the behavior of laboratory tests and proved to be a powerful tool for designing experiments as well as for interpretation of results.

7 Acknowledgement

The numerical simulation utilized the research results of the project supported by the Czech Science Foundation under Grant 16-04132S “Epistemic uncertainty of crack models in reinforced concrete structures”.

Thanks to ramsetreid (ITW Australia) for their continued support of our research efforts to endeavour to find innovative construction solutions.
References:


NUMERICAL ANALYSIS OF SEISMIC PERFORMANCE OF BEAM-TO-COLUMN POST-INSTALLED REBAR CONNECTIONS PRE-TEST SIMULATIONS

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ABSTRACT

Seismic strengthening and retrofitting of reinforced concrete structures frequently requires increased sizes (and reinforcement) of the existing beams. Functional changes during the service life of a structure can also require addition of beams to an existing structure. The efficiency of the added beams in carrying the acting forces depends on the effectiveness of the connection between the new beams and the existing structure (columns, beams and walls). To form a connection between the existing structural member and the beam, post-installed rebar are required. The behavior of these connections can be visualized either from the point of view of the reinforced concrete theory, which requires long bonded lengths (due to equivalence with cast-in reinforcement) or from the point of view of bonded anchor design that requires relatively short bond lengths. Approvals for application of post-installed reinforcing bars under seismic loads are available in US. In Europe there are no such provisions.

In this work, numerical simulations are performed to investigate the behavior of beam-column connections using post-installed rebar. The dependence of the connection behavior on failure modes is demonstrated using 3D finite element (FE) analyses. The method used for FE modelling has been validated with experiments available in literature. It is highlighted that in framework of an approach that is a function of the governing failure modes, it is possible to give consideration to the higher bond strength of post installed rebar connection for beam-column-connection applications. For further development of such an approach valid over a wide range of parameters, a proposal for performing certain benchmark experiments is also presented. Pre-test simulations of the proposed experiments are included.

Abbreviations Used:

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>CI</td>
<td>Cast in</td>
</tr>
<tr>
<td>CIS</td>
<td>Cast in Straight</td>
</tr>
<tr>
<td>PI</td>
<td>Post Installed</td>
</tr>
<tr>
<td>BCJ</td>
<td>Beam column joint</td>
</tr>
<tr>
<td>NSD</td>
<td>Non-seismically detailed</td>
</tr>
</tbody>
</table>

1 Introduction

Seismic retrofitting of reinforced concrete (RC) frame structures frequently requires increasing of member sizes for beams, slabs, walls or columns or addition of new members for a better lateral load
transfer mechanism. These modifications impose a requirement to establish a reliably safe connection with the existing structural members so that the intended transfer of forces is practically realized. Post-installed rebar offer a fast and convenient solution for establishing such connections. For gravity and normal service load applications EOTA-TR-023 (2006)\(^1\) provides the necessary assessment criteria which allows the use of post-installed rebar for connections. Design provisions in EN-1992-1-1 (2004)\(^2\) and ACI-318-14 (2014)\(^3\) allow the usage of post-installed rebar connections by considering them equivalent to cast-in connections provided the assessment criteria is satisfied for other than seismic applications. Although some post installation systems can offer significantly superior bond performance as compared to normal cast-in rebar, there exist no provisional guidelines to obtain benefit from this higher bond strength in design. Approvals\(^4\) for application of post-installed reinforcing bars under seismic loads are available in US. In Europe there are no such provisions. The non-availability of the provisional guidelines is largely because the behavior of post-installed systems is product dependent\(^5\), which makes it difficult to generalize a theory for such systems. Furthermore, seismic behavior of post-installed anchor systems is more complex because of the simultaneous load and crack cycling taking place at the anchor locations\(^6,7\), since unlike cast-in situations, it is not possible to provide supplementary reinforcements in a straightforward manner.

Notable work towards development of the design provision for post-installed rebar connections have been done by Spieth (2002)\(^8\). The behavior of post installed rebar under cyclic loads was investigated by Simons (2007)\(^9\). Observations indicate that the behavior of post installed rebar under cyclic load depends on the failure mode under monotonic loads\(^10\). Performance of post installed rebar for column-to-foundation connections was investigated through a series of experimentation by Mahrenholtz et al. (2014)\(^11\) and Herzog (2015)\(^12\). It was concluded that for cyclic loads, the behavior of post-installed connections with products which satisfy the assessment criteria\(^1\) can be considered to be equivalent to the cast-in connections with straight rebar. The behavior of these connections can be visualized either from the point of view of the reinforced concrete theory, which requires long bonded lengths (due to equivalence with cast-in reinforcement) or from the point of view of bonded anchor design that requires relatively short bond lengths. Attempts to harmonize these two design practices have also been made\(^12\). Development of strut-and tie method (STM) for design of post-installed rebar connection have been reported\(^13\) and the applicability was demonstrated by performing experiments on wall-to-foundation connections\(^14\).

Certain post-installed rebar systems have significantly higher bond strength characteristics as compared to cast-in rebars. In an experimental study\(^15\), it was shown that the behavior of structural members connected using post-installed anchor systems with low embedment depth can give an equivalent performance to the conventional reinforcement anchorage with bends; provided, the concrete cone formation is avoided. Comparative behavior of post installed rebar with the cast-in straight connections for similar embedment depth performed for wall-to-foundation connection\(^14\) show that the ductility behavior with post-installed (PI) system is better than cast-in straight (CIS) rebar. The study also highlights the product dependent behavior of post-installed rebar systems. Recent studies\(^16,17\) on the comparative behavior of CIS and PI rebars and concluded that the governing failure mode is the deciding factor, whether PI is better than or equivalent to CIS for any given set of parameters.
The studies of PI rebar connections on a structural subassembly level are primarily available for column-to-foundation and wall-to-foundation connections. Studies on employment of PI rebar for a beam column joint (BCJ) are not available in literature. Extensive experimentation and numerical data on seismic behavior of BCJ for cast-in (CI) connections of different configuration and detailing have been reported in the literature. From the viewpoint of seismic retrofitting, it is desirable to consider studies of non-seismically detailed (NSD) structural sub-assemblies. For NSD BCJ, the occurrence of joint shear failure in the failure hierarchy sequence is known to be the most critical factor in the process of re-evaluation of existing structures. For possible feasibility of installing a new beam with PI anchorage in an existing NSD column, it is essential to have an idea of the comparison of behavior of PI BCJ with conventional CI BCJ under the framework of governing failure modes. As a first step in this direction, preliminary parametric finite element analyses for a comparison of CI BCJ and PI BCJ behavior for a joint detailing designed to have a joint shear failure is considered in the present work. A proposal for an experimental program is provided in order to ascertain the actual behavior of PI BCJ.

2 Failure Modes in a NSD BCJ

Literature survey of numerous experimental and numerical studies on the behavior of non-seismically detailed beam column joints with cast in anchorage of different configurations is available. The possible failure modes for a NSD CI BCJ are:

i. Joint shear failure,
ii. Beam flexure failure,
iii. Column flexural failure,
iv. Beam shear failure,
v. Column shear failure.

From seismic considerations, beam flexure failure is a desired governing failure mode. The seismic design of a joint should be such that all other failure modes are arrested. For a NSD BCJ, joint shear failure typically results because of the absence of stirrups (confinement) in the joint core. Depending on the geometry of the joint, material properties of concrete and steel and the reinforcement in the beam, it is possible to determine the loads corresponding to the joint shear failure.

In the event of use of PI rebars for anchorage of the beam main bars in the joint core, it is essential to consider the bond-splitting failure in addition to the above five failure modes. High end adhesive systems typically employed for PI rebars have much higher bond strength as compared to CI connections. Due to the limited width of the column section, and depending on the spacing of the beam main bars, there is a possibility of variation in the bond-splitting mode behavior. Furthermore, the effect of bond splitting on the joint shear capacity is also possible.

3 FE Modelling Approach and Validation

An experiment on NSD CI BCJ with bent-in reinforcement configuration on top and straight reinforcement on the bottom (Figure 1) was considered as benchmark for validation of the FE analysis approach—Genesio and Sharma (2012) (JT31 Configuration). FE analysis was performed using the software MASA. A half-model was prepared so that the advantage of symmetry
could be utilized. Column and beam concrete are modelled by solid elements. The microplane model with relaxed kinematic constraint \(^{20}\) is used as the constitutive law for concrete. All reinforcements were modelled by bar elements. Zero length bond elements were defined in the joint in order to connect the reinforcement bar element to the concrete. In all other regions, a direct connection between the reinforcement and concrete simulating perfect bond was employed. Trilinear stress-strain curve was used to define the uniaxial constitutive law for reinforcement while the von-Mises stress criterion was used to define yielding of the reinforcing bar. A multi-linear bond stress-slip law was assigned to the zero length bond elements.

![Figure 1: Geometry of joint tested by Genesio\(^{18}\) (JT31 configuration)](image1)

3.1 Validation with test by Genesio (2012)\(^ {18}\)

The geometry of the joint with JT31 configuration as tested by Genesio\(^ {18}\) is shown in Figure 1. A FE model as described earlier for this joint was prepared and cyclic analysis was performed. A comparison of the results of experiment and analysis in terms of the load displacement behavior is shown in Figure 2.

![Figure 2: Comparison of load-displacement behavior of experiment\(^ {18}\) with analysis](image2)
Figure 3: Failure mode in terms of crack pattern (a) experiment\textsuperscript{18}, (b) Analysis loading upwards and (c) Analysis loading downwards

A comparison of failure mode of experiment (Figure 3a) with analysis for the downward loading with bent in bar in tension is shown in Figure 3c and for the upwards loading with the straight bar in tension is shown in Figure 3b. The FE approach is seen to predict the load displacement behavior as well as the cracking pattern as observed in the experiment in a sufficiently good manner.

For NSD BCJ with the beam main bars anchored as PI rebars, presently no experimental data is available. Therefore a validation of numerical FE approach for this case is not possible at this stage. However, as a first step towards establishing behavior of PI rebars applied to beam to column connections, some parametric FE analysis is performed and an experimentation program is proposed.

4 Parametric FE analysis

As a first step towards comparison of behavior of NSD BCJ with PI rebars connection, FE analyses of three beam column joints with identical beam and column geometry was performed. All three joints had a 350mm x 350mm column with span of 3m, and a 300mm x 450mm beam. Beam reinforcement with equivalent of 4 bars of 20mm at top and bottom has been selected in order to ascertain the joint shear failure mode. More details of the configuration of joints of three specimens are as followed:

(i) CI-4-20-NSD: Cast in specimen with bent in bar on top and straight bars at the bottom (embedment 300mm) in the joint core. 4-20mm bars at top and bottom for the beam.
Monotonic analyses with the aforesaid mentioned modelling approach was performed for all specimens. The predictions of ultimate loads in terms of the moment in the beam section at the column interface, and the expected failure modes for the three specimens are as followed:

(i) CI-4-20-NSD: For downward loading of the beam with the bent in bars in tension, the failure mode is expected to be joint shear with a crack pattern as shown in Figure 4a. The damage is expected to be concentrated along a primary diagonal crack. The joint failure is expected to take place at a beam moment of 148kNm at the column face. For upwards loading of the beam with the straight bars in tension, the failure is expected to take place at a beam moment of 102kNm at the column face. The mode of failure is again expected to be joint shear; however, the damage is expected to be distributed in several diagonal cracks along the straight embedment of the bar as indicated in Figure 4b.

(ii) PI-4-20-NSD: The specimen is expected to have a similar magnitude of ultimate load for both directions of loading. The ultimate beam moment is expected 98kNm. It is expected that this specimen exhibits several diagonal cracks along the bar embedment. Additionally, it is expected that a dominant splitting-cum-column flexure crack develops along the embedded bar owing to the extensive splitting forces resulting from the higher bond strength of the adhesive mortar. In the post peak region the specimen is expected to exhibit cone failure cracks.

(iii) PI-2-28-NSD: The specimen is expected to have a similar magnitude of ultimate load for both directions of loading. The ultimate beam moment is expected 118kNm. It is expected that this specimen exhibits several diagonal cracks along the bar embedment. In the post peak region the specimen is expected to exhibit cone failure cracks.
5 Conclusions

Application of post installed rebars for non-seismically detailed existing frames is considered in the present work. Since no experimental data on this subject is available to date, as an initial step, three non-seismically detailed joints have been analyzed. A finite element modelling approach for non-seismically detailed joints used in the present work was validated with experiments performed with conventional cast-in rebars. Using this approach, preliminary parametric analysis for NSD BCJ specimens with post installed rebars targeted to fail in joint shear failure mode was performed. The expected results from the analyses have been presented in this paper.

It should be noted that presently no experimental data on behavior of beam-column joints with post installed reinforcement is available. It is hereby proposed that experiments with the joint configurations considered in the present work be performed so that the actual behavior BCJ with PI rebars used for anchorage of beam main reinforcement can be ascertained.

References:


3. ACI-318-14. Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary. American Concrete Institute; 2014.


DETERMINATION OF MATERIAL PROPERTIES FOR STRUCTURAL APPLICATIONS

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1Christian Doppler Laboratory LiCRoFast, Institute of Structural Engineering, University of Natural Resources and Life Sciences, Vienna, Austria
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ABSTRACT

The safe and accurate prediction of the long-term performance of concrete structures is an important task. As a result, it is fundamental to understand phenomena, which influence the load bearing capacity in course of time. The main source of concrete aging is the ongoing hydration, which is influenced by the concrete mix design as well as the environmental conditions. Current design codes suggest empirical aging functions for some of the material properties, in order to determine strength levels at any given age. However, these equations assume a spatially homogeneous material and express the development of mechanical properties in terms of time, instead of actual hydration degree (curing level), which is a spatially variable temperature dependent quantity. This approach can be problematic for large concrete structures, which exhibit a large gradient of temperature and humidity resulting in a gradient of maturity. Of course, this is an important topic, considering that the assumed material properties, which are tested on small scale specimen, may differ from the real properties in a structural member. Therefore, the real evolution of material properties, depending on the curing conditions is studied based on an example of fastening applications by means of chemo-mechanical simulations and experiments. The aim of this paper is to compare standard specimens undergoing different curing protocols and anchor tests at 28 days, and in the course of time. As a result, a comparison between the different states of hydration is given for the studied test member and the companion specimen.

1 Introduction

Due to growing demand in both design and performance of state of the art concrete buildings, the applications of fastenings become more and more relevant. Naturally, it is important to fully understand not only the structure but also the single components of fastening systems. Herein, one of the most important components is concrete. Nevertheless, concrete is a complex material and has to be understood in depth in order to guarantee durability and high safety levels.

As a result it is important to study the long time performance of the material which builds the base for an anchorage system. However, the long time performance of concrete depends on different phenomena which have to be captured in the prediction of mechanical properties. As widely known, the mechanical strength of concrete such as compressive strength can be directly linked to the degree...
of hydration in a concrete member. Anyhow, the hydration of concrete highly depends on the mix
design and the given environmental conditions and as result is spatially variable.

For approval testing of fastening products, ETAG\textsuperscript{1} suggests to test compressive strength on cube
specimens with a side length of 150 mm or cylindrical specimen with a diameter of 150 mm and a
height of 300 mm. The curing condition of these specimens should follow the same curing condition
as for the test member in which the anchors are installed. The test members shall be cured indoors
for at least 7 days. Afterwards outside storage is possible, if protection from direct sun, frost or rain
is given. Further ETAG suggests, that the small specimens should represent the material property
of the test members. If in doubt, cores should be drilled and tested for determining the representative
material property.

This paper studies the agreement between standard control specimens undergoing different curing
protocols and test members. The degree of curing is determined by the hygro-thermal chemical
model, developed by Di Luzio et al.\textsuperscript{2,3}, after careful calibration based on calorimetric data.

2 Numerical approach (HTC model)\textsuperscript{2,3}

In the hydration process of concrete water is consumed, which leads to a volume reduction, better
known as autogenous shrinkage. Additionally, a volume reduction is caused by waterloss due to
diffusion, which highly depends on the environmental conditions such as humidity and temperature
drying shrinkage). Further, the temperature rises during the initial phase of curing, due to the
chemical reaction. The speed of the chemical reaction as well as the transport processes highly
depend on the temperature. On the other side, the availability of water influences also the hydration
reaction which, under a certain level, stops completely. In order to accurately predict the long-term
behavior of concrete, it is essential to have a numerical framework, which is able to capture all these
highly-coupled phenomena.

The hygro thermal chemical model describes all mentioned phenomena by solving the mass and heat
balance equations as well as equations for the chemical reaction and the moisture transport
(diffusion). The time evolution of temperature and humidity are given by the following equations

\[
\nabla \cdot (D_h \nabla h) - \frac{\partial w_e}{\partial h} \frac{\partial h}{\partial t} \dot{a}_c - \frac{\partial w_e}{\partial a_s} \dot{a}_s - \dot{w}_n = 0 \tag{1}
\]

\[
\nabla \cdot (\lambda_c \nabla T) - \rho c_t \frac{\partial T}{\partial t} + \dot{a}_c c \tilde{Q}_e^\infty + \dot{a}_s s \tilde{Q}_s^\infty = 0 \tag{2}
\]

in which the permeability $D_h$, the relative humidity $h$, the evaporable water $w_e$, the cement hydration
degree $a_c$, the silica fume reaction $a_s$, the non-evaporable water $w_n$, the heat conduction coefficient
$\lambda_c$, the temperature $T$, the concrete density $\rho$, the isobaric heat capacity $c_t$, the cement content $c$, the
silica fume content $s$, the cement hydration enthalpy $\tilde{Q}_e^\infty$ and the silica fume enthalpy $\tilde{Q}_s^\infty$ are taken
into account.

The results of the simulations are spatial fields of evolving cement hydration degrees, temperature
and humidity, depending on the prescribed environmental boundary conditions.
3 Input parameters

3.1 Used concrete mix

The numerical study of the influence of different curing conditions is based on a normal strength concrete with a strength class of C25/30. The mix design is summarized in Table 1.

Table 1: Mix design of used concrete.

<table>
<thead>
<tr>
<th></th>
<th>amount</th>
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<tbody>
<tr>
<td>w/c</td>
<td>0.63</td>
<td>-</td>
</tr>
<tr>
<td>Density</td>
<td>2453</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Silica Fume content</td>
<td>0</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Aggregate content</td>
<td>1907.6</td>
<td>kg/m³</td>
</tr>
<tr>
<td>Cement type</td>
<td>CEM II 42.5 N</td>
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</tr>
</tbody>
</table>

Further the hydration input parameters are calibrated based on a calorimeter test for the given cement type. Additionally, the permeability of the concrete is calibrated based on experimental drying tests of the same concrete.

3.2 Specimen dimensions

The test member for the installation of anchors is assumed to be a slab with the dimensions of 100x250x30 cm. The small-scale specimens are following the ETAG5 suggestions: Cube specimen with a side length of 150 mm and cylinders with a diameter of 150 mm and a height of 300 mm.

3.3 Curing conditions

For this study, the test member (slab with anchors) is stored at a mean temperature of 12°C and a humidity of 60%. Further, different curing scenarios are simulated for small specimens (cubes and cylinders). Following current testing standards for the determination of material properties, specimens cured under water or in a moist atmosphere at a constant temperature of about 20°C are required for testing compressive strength. However, ETAG, suggests to determine the related material property on specimens that undergo the same treatment as the slabs. The third option is curing the small-scale specimens with unlimited water supply but following the same temperature history as the slabs. Based on these different approaches the curing conditions listed in Table 2 are investigated.

Table 2: Overview study comparison.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Locations</th>
<th>Temperature [°C]</th>
<th>Humidity [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab</td>
<td>Outside</td>
<td>12</td>
<td>60</td>
</tr>
<tr>
<td>Cube</td>
<td>Outside</td>
<td>12</td>
<td>60</td>
</tr>
<tr>
<td>Cube</td>
<td>Outside</td>
<td>12</td>
<td>100 (water)</td>
</tr>
<tr>
<td>Cube</td>
<td>Lab</td>
<td>23</td>
<td>100 (water)</td>
</tr>
<tr>
<td>Cylinder</td>
<td>Outside</td>
<td>12</td>
<td>60</td>
</tr>
<tr>
<td>Cylinder</td>
<td>Outside</td>
<td>12</td>
<td>100 (water)</td>
</tr>
<tr>
<td>Cylinder</td>
<td>Lab</td>
<td>23</td>
<td>100 (water)</td>
</tr>
</tbody>
</table>
4 Discussion of results

Since current design codes are referring in most of the cases to the 28 day strength, also the curing state is analyzed for this age. Furthermore, the time development of the cement hydration degree for different scenarios is discussed.

Figure 1 shows the distribution of the cement hydration degree at a curing age of 28 days at an ambient temperature of 12°C and 60% ambient humidity. As can be seen, a strong gradient of the hydration degree is given towards the edges, where the influence of the boundary conditions is more pronounced. Towards the inside, the gradient decreases and in center of the specimen the highest degree of 0.65 is determined. Furthermore, Figure 2 shows the evolution of the cement hydration degree in the middle of the slab. Naturally, the hydration degree increases more rapidly in the beginning and slows down at approximately 28 days. This behavior is also described by empirical aging functions suggested by the design codes.

Considering Figure 3, the hydration degree in a cube specimen, dry cured (12°C, 60% RH) can be seen. Similar to the slab, a gradient from the outside to the inside can be noticed, but the maximum reaction degree reached is lower than for the slab. Further, a difference in the evolution of hydration is given. Comparing the slab with the dry cured cube, it is obvious that the hydration of the slab is faster than for the cube. The reason is the higher temperature and water availability inside the slab. Consequently, the material property of a dried small scale cube is different than the one inside a slab at a location where an anchor typically may be installed. Further, the failure mechanism of the small specimen during the material test has to be taken into an account. During a cube compression test, all the material towards the surface that exhibits a large gradient in maturity is involved, while the performance of e.g. a headed stud is almost exclusively determined by the material properties inside the slab, at least up to peak.

Figure 4 illustrates the simulation results of a specimen which is moist cured at a constant temperature of 23°C (lab condition). In this case the hydration is faster, due to the increased temperature and a spatially constant hydration degree is obtained (due to the availability of water during the hydration). The development of the hydration degree is closer to the one given in the slab and any form of shrinkage damage is avoided.

The curing condition, which represents the properties of the slab the most accurately, is a small-scale specimen, cured in water or a high humidity environment but undergoes the same temperature history as the test member. This can be seen in Figure 5. Herein, the hydration degree is the same for both the small specimen and the slab (0.65), but also the evolution in time is almost identical.

The results for the cylindrical specimen can be seen in Figure 6 to 8. As for the cubes, the failure mechanism has to be considered for the dry cured case, which shows a gradient from the surface towards the inside of the specimen. However, the results are qualitatively similar to those obtained for the cube specimen and are not discussed further.
Figure 1: Cement hydration degree at 28 days inside slab.

Figure 2: Evolution of cement hydration degree inside slab.
Figure 3: Dry cured cube: Left: Cement hydration degree at 28 days; Right: Evolution of cement hydration degree.

Figure 4: Moist cured cube (23°C): Left: Cement hydration degree at 28 days; Right: Evolution of cement hydration degree.
Figure 5: Moist cured cube (12°C): Left: Cement hydration degree at 28 days; Right: Evolution of cement hydration degree.

Figure 6: Dry cured cylinder: Left: Cement hydration degree at 28 days; Right: Evolution of cement hydration degree.
Figure 7: Moist cured cylinder (23 °C): Left: Cement hydration degree at 28 days; Right: Evolution of cement hydration degree.

Figure 8: Moist cured cylinder (12 °C): Left: Cement hydration degree at 28 days; Right: Evolution of cement hydration degree.
5 Conclusion

After simulating different curing conditions for fastening test members and the related standard specimens for the determination of material properties the following conclusions can be drawn:

- The hydration degree is spatially variable for dry cured specimens
- The moist cured specimens have a constant hydration degree over the whole cross-section, which is beneficial considering the failure mechanisms in compressive strength tests
- The development of the hydration degree depends on the ambient temperature and humidity history
- For this numerical investigation, the condition in the middle of a slab, is best characterized by a specimen (cube or cylinder) that is moist cured with the same temperature regime
- There is no significant difference for the cube and cylindrical specimen.

6 Acknowledgement

The financial support by the Austrian Federal Ministry of Economy, Family and Youth and the National Foundation for Research, Technology and Development is gratefully acknowledged, as is the additional support by our industrial partners.

The computational results presented have been achieved using the Vienna Scientific Cluster (VSC).

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NUMERICAL AND EXPERIMENTAL EVALUATIONS OF
INFLUENCE OF MEMBER THICKNESS, ANCHOR HEAD SIZE, AND
SURFACE REINFORCEMENT ON TENSILE BREAKOUT CAPACITY
OF ANCHOR BOLTS

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ABSTRACT

The influence of member thickness, size of anchor head, and orthogonal surface reinforcement on the tensile breakout capacity of cast-in-place headed anchors in uncracked concrete was studied both numerically and experimentally. The aim of this paper is to form a background for developing improved methods for the design of new fastenings as well as the assessment of current anchorages in practice. For this purpose, anchor bolts at various embedment depths \((h_{ef}=50–500 \text{ mm})\) were simulated in plain and reinforced concrete members of various thicknesses \((H=1.5–5.0\times h_{ef})\). Three different head sizes of anchor bolts (i.e. small, medium and large) were also considered at each anchor embedment depth. Furthermore, to verify the numerical findings, a series of anchor pullout tests were carried out at which the testing parameters were similar to those in the numerical study.

Numerical and experimental results show that the tensile breakout capacity of anchor bolts increases by increasing the member thickness or if surface reinforcement is present. The anchorage capacity further increases with increasing the anchor head size. The anchorage behavior becomes ductile by increasing member thickness or by having surface reinforcement, whereas it becomes stiff and more brittle by increasing the size of anchor head. To account for the influence of member thickness, size of anchor head, and orthogonal surface reinforcement on the tensile breakout capacity of headed anchors, the CC method was modified and extended by incorporating three modification factors.

1 Introduction

Various anchorage systems including both cast-in-place and post-installed anchors have been developed for fastening both non-structural and structural components to concrete structures. Cast-in-place anchors have been used ever since reinforced concrete was introduced around 1900. Post-installed anchors started to be used in the 1960ies in concrete structures with the advances in drilling technology. An overview of the anchorage technology is given in Eligehausen et al.\(^1\).

A need for more flexibility in the design of new structures as well as for strengthening of existing concrete structures has led to an increasing use of various metallic anchors in practice. Although
every year millions of fasteners are used in the construction industry around the world, the state of knowledge about the fastening technology is still poor. In a sustainable society, buildings and structures usually need to be adjusted to new demands from time to time. Often loads on structures have to be increased to comply with new demands, which require upgrading not only the structural components but also the structural connections. From the structural connection point of view, a specific question that always has to be addressed is whether the current fastenings are safe enough for the intended increased load or if they must be replaced or upgraded. The current design models are generally believed to be conservative, but it is not very clear to what extent. These issues call for more refined and reliable models that can be used in the design of new fastenings as well as the assessment of current anchorages in practice.

The current design models for cast-in-place mechanical anchors are simple and based on the simplifying assumptions that the size of anchor head, orthogonal surface reinforcement in concrete walls and slabs, and thickness of concrete member have negligible influence on the anchorage capacity and performance. In general, concrete materials in the anchoring zone of an anchor bolt experiencing tensile load are often subjected to two stress fields: a local stress field, resulting from the interaction between the anchor bearing head and the concrete, and a global stress field resulting from concrete-member bending induced by the anchor transverse load. Such bending induces concrete tensile stresses, which may result in concrete cracking. The concrete tensile stresses can be reduced by increasing the global bending stiffness of the concrete member which basically depends on the properties of concrete, the thickness of concrete member, and the amount of surface reinforcement. Therefore, these parameters may affect the tensile breakout resistance of anchors. In addition, the size of anchor-head has an effect on the local stresses in the vicinity of the anchors: the local concrete stresses in the anchor bearing area decrease with increasing head size.

To refine the current design models for cast-in-place mechanical anchors and to better understand the influence of member thickness, size of anchor head, and orthogonal surface reinforcement on the anchorage capacity and performance, an extensive numerical Finite Element (FE) study and a series of experiments were carried out. In the present paper, a short review of the current design models and a summary of the numerical and experimental studies are presented.

2 Review of Current Design Methods

The mean tensile breakout capacity of anchors can be calculated by the Concrete Capacity (CC) method\(^2\). This method was incorporated into several design standards in the US, such as ACI 349\(^3\) and ACI 318\(^4\), and also into various design-oriented documents in Europe (e.g. CEB\(^5\) and CEN/TS 1992-46), and internationally in the fib Bulletin 58\(^7\). Based on the CC method, the mean tensile breakout capacity of a single anchor bolt in un-cracked concrete, far from concrete free edge/s and/or adjacent anchor/s, can be evaluated using (Eq. 1).

\[ N_{u,m} = 16.8 \sqrt{f_c h_{ef}^{1.5}} \]  

(1)

where \(N_{u,m}\): mean concrete cone breakout capacity of a single anchor [N], \(f_c\): concrete cylinder compressive strength [MPa], \(h_{ef}\): anchor effective embedment depth [mm]. Equation (1) is conservative for large embedment depths, because it takes into account the maximum size effect
according to linear fracture mechanics. This has been previously demonstrated by Ozbolt et al.\textsuperscript{8} by numerical simulations. ACI 349\textsuperscript{3} and ACI 318\textsuperscript{4} allow the use of a modified CC method for large anchors (i.e. $h_{ef} \geq 279.4$ mm). The modified CC method uses an exponent of $(5/3=1.667)$ instead of 1.5 for the effective embedment depth of deep anchors and appropriately changes the leading coefficient. Based on this model, the mean concrete cone capacity of a single anchor bolt under tension load in uncracked concrete can be determined as below:

\begin{align}
N_{u,m} &= 16.8\sqrt{f_{c}h_{ef}^{1.5}} \quad \text{for } h_{ef} < 280 \text{ mm} \tag{2a} \\
N_{u,m} &= 6.585\sqrt{f_{c}h_{ef}^{5/3}} \quad \text{for } 280 \text{ mm} \leq h_{ef} \leq 635 \text{ mm} \tag{2b}
\end{align}

Equations (1) and (2) for cast-in-place mechanical anchors predict the mean values of the failure loads measured in a large number of anchor pullout tests at various embedment depths. The tested anchors were embedded in un-reinforced and lightly-reinforced concrete members of various thicknesses and also had various head sizes. The ratios of measured concrete cone breakout capacities to the values predicted by the CC method (Eq. 1) as a function of anchor embedment depth for 288 pullout tests from literature\textsuperscript{9} are shown in Fig. 1(a).

As the tested headed anchors had different head sizes, the concrete in the vicinity of the anchor heads experienced various bearing pressures. The ratios of bearing pressure under the anchor head at peak load to the concrete compressive strength ($p_b/f_c$) for the tested headed anchors are plotted as a function of anchor embedment depth in Fig. 1(b).

![Figure 1](image.png)

Figure 1: (a) ratio of measured concrete cone failure loads to values predicted according to (Eq. 1) for tested anchor bolts as a function of anchor embedment depth, and (b) ratio of bearing pressure under head of tested anchors at peak load to the concrete cylinder compressive strength as a function of anchor embedment depth (test data compiled and re-evaluated from Zhao\textsuperscript{9})

The tested short anchors ($h_{ef} \leq 100$ mm) had relatively large heads as standard headed studs were used at small embedment depths (the mean bearing pressure under the anchor head at peak load ($p_b$) was approximately $4.6f_c$), whereas the tested large anchors ($h_{ef} > 100$ mm) had relatively small heads (the mean bearing pressure under the anchor head ($p_b$) was approximately $14.8f_c$). This figure explicitly indicates that there is no consistency in respect to the size of anchor head in the development of the CC method, as the bearing pressure under the anchor head was considerably variable for the short and deep anchors. In fact, a reduction in the bearing area of the short anchors ($h_{ef} \leq 100$ mm) may lead
to a decrease of their tensile resistances and also an increase in the bearing area of deep anchors ($h_{ef} > 100$ mm) may cause an increase of their breakout capacities.

As in common practice, anchor bolts with various head sizes are often installed in plain and reinforced concrete members of various member thicknesses, in some cases the CC method may underestimate the anchorage capacity (that may be the case for deep anchor bolts with large heads in a thick reinforced concrete member) or even overestimate the anchorage capacity (that can be the case for short anchor bolts with small heads in a relatively thin unreinforced concrete member). It is believed that it is possible to refine the CC method if the influence of member thickness, amount of surface reinforcement and size of anchor head is taken into account. However, this requires performing systematic experimental and numerical studies to individually evaluate the influence of each parameter on the anchorage capacity and performance.

3 Numerical Study

For the purpose of this research an extensive numerical study was carried out. The numerical analyses were performed through a three-dimensional FE code (i.e. called MASA). The employed constitutive material model for concrete was based on the microplane constitutive law. To systematically evaluate the influence of member thickness, anchor head size and orthogonal surface reinforcement on the tensile breakout capacity of headed anchors, three simulation series were considered. In series (i), headed anchors at various embedment depths ($h_{ef} = 50–500$ mm) were simulated in unreinforced concrete members of various thicknesses ($H = 1.5–5.0 h_{ef}$). In series (ii), headed anchors at various embedment depths ($h_{ef} = 50–500$ mm) were considered to have various head sizes (i.e. small, medium and large heads). For this simulation series, all concrete members were unreinforced and the member thickness was ($H = 3.0 h_{ef}$). In series (iii), headed anchors at various embedment depths ($h_{ef} = 50–500$ mm) were simulated in orthogonally reinforced concrete slabs of various thicknesses ($H = 1.5–3.0 h_{ef}$). In addition, the concrete slabs were considered to be lightly-reinforced ($\rho \approx 0.3\%$) and also over-reinforced ($\rho > 0.5\%$) to evaluate the influence of reinforcement-content on the anchorage capacity and performance.

In all simulation series, the length ($L$) and width ($W$) of concrete slabs, for all embedment depths of anchors, were scaled systematically and were proportional to the anchor embedment depth ($L = W = 6.0 \cdot h_{ef}$). The typical geometry of specimens and the discretized FE model are shown in Fig 2. In series (ii), the diameter of the anchors head was set so that the bearing pressure under the head at anchors peak load would be almost constant for all investigated embedment depths of anchors. The peak load for all anchors was predicted using the CC method (Eq. 1). The ratios of bearing pressure under the head at the anchors peak load to the concrete compressive strength ($p_b/f_c$) for the small-, medium- and large-headed anchors were approximately 20, 11 and 4, respectively.

Before simulating all FE models of this study, FE models were initially calibrated and verified against anchor pullout tests performed previously by Nilsson et al. The concrete properties for all FE models were as follows: uniaxial compressive strength $f_c = 28$ MPa, uniaxial tensile strength $f_t = 2.2$ MPa, Young’s modulus $E_c = 35000$ MPa, fracture energy $G_f = 70$ N/m, and Poisson’s ratio $\nu_c = 0.18$. The behavior of steel in the anchor was assumed to be linear elastic with Young’s modulus $E_s = 210000$ MPa and Poisson’s ratio $\nu_s = 0.33$. Anchor pullout loading was performed under displacement control by applying incremental deformations on the top of the anchor shaft.
Due to a symmetrical geometry, only a quarter of the specimens were simulated to save the time of analyses (double symmetry boundary conditions were defined). For a complete description of the FE simulation and verification procedure see Nilforoush et al.\textsuperscript{13}.

\begin{figure}[h]
\centering
\includegraphics[width=0.8\textwidth]{figure2}
\caption{(a) Typical geometry of specimens, and (b) FE discretized model (one quarter model)}
\end{figure}

4 Experimental Study

To verify the numerical findings and better clarify the influence of member thickness, size of anchor head, and orthogonal surface reinforcement on the anchorage capacity and performance, a supplementary experimental study was carried out\textsuperscript{14}. A total of nineteen headed anchors cast-in unreinforced and reinforced concrete slabs were tested under monotonic tensile loading. Due to financial limitations, only one anchor embedment depth was tested ($h_{\text{ef}}=220$ mm). Like the numerical study, three test series were considered in which the testing parameters were the same as the numerical study. The test setup is schematically shown in Fig. 3.

\begin{figure}[h]
\centering
\includegraphics[width=0.8\textwidth]{figure3}
\caption{A schematic view of test setup}
\end{figure}

The length and width of concrete blocks for all specimens were identical ($L=W=1300$ mm), whereas the height of concrete blocks varied ($H=330$, $440$ and $660$ mm). The reinforced slabs were lightly-reinforced ($\rho\approx0.3\%$). For anchor tension loading, the vertical reaction was taken up by a stiff circular steel ring with an inner diameter of $L_{\text{sup}}=880$ mm. The slabs were cast using a normal-weight concrete of class C30/37 made of crushed aggregates (concrete compressive and tensile splitting strengths at time of anchor pullout loading were $f_c=33.2$ MPa and $f_{t,\text{sp}}=3.2$ MPa, respectively). The
headed anchors consisted of steel rods of grade 10.9 (yield strength $f_{yk}=900$ MPa and ultimate strength $f_{uk}=1000$ MPa). The anchor tension loading was displacement-controlled by applying incremental deformations on the top of anchor shaft. The load was applied by means of a 100-ton hollow cylinder hydraulic jack. The time-deformation relation was kept approximately linear.

5 Results and Discussion

A selection of the results of this study is presented in the followings and additional results are given in Nilforoush et al.\textsuperscript{10,11,14}.

5.1 Influence of member thickness

The numerical and experimental results of anchor bolts in unreinforced concrete members of various thicknesses (i.e. series (i)), showed that the tensile breakout capacity of anchors increases up to 20% with increasing the member thickness. The numerically obtained load-displacement curves and post-peak crack patterns for an anchor bolts at 200 mm embedment depth in unreinforced concrete members of various thicknesses are shown in Fig. 4 (a) and (b), respectively. The anchorage is governed by concrete bending/splitting cracks in thin unreinforced concrete members ($H<2.0\cdot h_{ef}$) while it is governed by concrete cone breakout in thicker members ($H\geq2.0\cdot h_{ef}$). That is the case for all simulated embedment depths of anchors.

![Figure 4](image)

Figure 4: Anchor bolts in unreinforced concrete members of various thicknesses: (a) load-displacement curves at simulations, (b) post-peak crack patterns at simulations, (c) load-displacement curves at tests, and (d) crack patterns at tests

These findings are in good agreement with the experimental results: see the load-displacement curves and crack patterns obtained at tests in Fig. 4 (c) and (d), respectively. The experimentally and numerically obtained influence of member thickness on failure loads agree quite well.
5.2 **Influence of size of anchor head**

The numerical and experimental results of anchor bolts with various head sizes showed that the anchorage capacity and stiffness increases by increasing the head size. However, the post-peak anchorage behavior becomes more brittle with increasing head size. The load-displacement curves and the post-peak crack patterns for an anchor bolt at 200 mm embedment depth with various head sizes (i.e. small, medium and large) are shown in Fig. 5 (a) and (b), respectively. The post-peak crack patterns shows that, irrespective of the head size, the anchorage is governed by concrete cone breakout failure. However, the average concrete cone angle, with respect to the loading direction, increases with increasing head size. In addition, the diameter of the cone at the concrete surface also increases by increasing head size. This finding strongly concurs with the test results: see the load-displacement curves and crack patterns obtained in tests in Fig. 5 (c) and (d), respectively.

![Simulation results](image)

**Simulation results** $h_{ef}=200$ mm

- Small head
- Medium head
- Large head
- CC Method

![Test results](image)

**Test results** $h_{ef}=220$ mm

- Small head
- Medium head
- Large head

Figure 5: Anchor bolts with various head sizes: (a) load-displacement curves at simulation, (b) post-peak crack patterns at simulation, (c) load-displacement curves at test, and (d) crack patterns at tests

It should be emphasized that for the large-headed anchors the formation of concrete cone cracks was hindered by the vertical support thereby changing the failure mode to bending cracking. This transition of failure mode happened after the peak load was reached and thus it does not seem to affect the anchorage capacity, however affects the post-peak anchorage behavior. This change of failure mode explains the very brittle post-peak behavior of the large-headed anchors. The numerical results showed an increase rate of up to 16% with increasing head size, while the experimental results showed a larger increase rate of approximately 34%.
5.3 Influence of reinforcement-content

The numerical and experimental results of anchor bolts in reinforced concrete members (i.e. series (iii)) showed that the anchorage capacity and ductility increases if a light amount of orthogonal surface reinforcement is present (i.e. $\rho=0.3\%$). The comparison of numerically obtained load-displacement curves of an anchor bolt at 200 mm embedment depth in unreinforced and reinforced concrete members of various thicknesses are shown in Fig. 6 (a). In this figure, from the left to the right, the member thickness increases from 1.5 to 3.0 times the anchor embedment depth. As the figure shows, the over-reinforced concrete does not improve the anchorage capacity and performance any further than the lightly-reinforced concrete. Furthermore, the favorable influence of orthogonal surface reinforcement on the tensile breakout capacity of anchor bolts decreases with increasing member thickness. This is the case for all simulated embedment depths of anchors. The load-displacement curves for tested anchor bolts in reinforced concrete slabs of various thicknesses are shown in Fig. 6 (b). As the figure shows, the experimental results confirm the tendency observed in the numerical study.

![Simulation results](image1)

![Test results](image2)

Figure 6: Comparison of load-displacement curves of anchor bolts in unreinforced and reinforced concrete members of various thicknesses: (a) numerical results, and (b) experimental results

The failure of all simulated and tested anchor bolts in reinforced concrete members was concrete cone breakout. In fact, the observed concrete bending/splitting failure in thin unreinforced concrete is prevented by a light amount of orthogonal surface reinforcement (see the numerically and experimentally obtained crack patterns in Fig. 7 (a) and (b), respectively).
Figure 7: Crack patterns for anchor bolts in reinforced concrete members of various thicknesses, (a) numerical results, and (b) experimental results

6 Proposals for the Refinement of CC Method

Based on the numerical results of anchor bolts at various embedment depths in unreinforced concrete members of various thicknesses it was found that Equation (1) overestimates the tensile breakout capacity of the short anchors ($h_{ef} \leq 100$ mm) in thin unreinforced members, whereas it underestimates the breakout failure load of deep anchors ($h_{ef} > 100$ mm) in thick unreinforced members (see ratios of calculated failure loads to values predicted by Equation (1) for simulated anchor bolts in unreinforced concrete members of various thicknesses as a function of anchor embedment depth in Fig. 8).

Figure 8: Ratios of calculated failure loads to values predicted by the CC method for the simulated anchor bolts in unreinforced concrete members of various thicknesses as a function of anchor embedment depth
In addition, the numerical results of anchor bolts at various embedment depths with various head sizes showed that the Equation (1) underestimates the failure load of headed anchors with large heads (i.e. corresponding to a bearing pressure lower than 11f_c under anchor head at peak load). Taking the influence of member thickness, anchor head size, and orthogonal surface reinforcement into account may help improving the current models for predicting the tensile breakout capacity of cast-in-place headed anchors.

As both the CC method (Eq. (1)) and the modified CC method (Eq. (2a)) are identical for anchor bolts with embedment depths shorter than 280 mm and both methods overestimate the numerically obtained failure load of short anchors in unreinforced concrete, it is recommended to use the modified CC method (Eq. (2b)), associated with the deep anchors (h_{ef} ≥ 280 mm), also for short embedment depths of anchors, as given in (Eq. (3)).

\[ N_{u,m} = 6.585 \sqrt{f_c(h_{ef})^{5/3}} \]  
Equation (3) is valid for single headed anchors with relatively small heads (p_b=15·f_c under the anchor head at peak load) in an uncracked unreinforced concrete member with a thickness of H=2.0·h_{ef}. Compared with equations (1) and (2), equation (3) predicts slightly lower capacities for short anchors and yields smaller deviations with the numerically obtained capacities over the entire investigated range of anchor embedment depths (h_{ef}=50–500 mm). Furthermore, to account for the influence of member thickness, size of the anchor head, and surface reinforcement on the anchorage capacity of headed anchors in uncracked concrete, Equation 3 is extended by incorporating three modification factors (i.e. \( \Psi_H \) for member thickness, \( \Psi_{AH} \) for anchor head size, and \( \Psi_{sr} \) for surface reinforcement), which are defined as follows:

\[ N_c = N_{u,m} \cdot \Psi_H \cdot \Psi_{AH} \cdot \Psi_{sr} \]

\[ \Psi_H = \left( \frac{H}{2.0 \cdot h_{ef}} \right)^{0.25} \leq 1.20 \]
\[ \Psi_{AH} = \left( \frac{A_b}{A_b^{\text{code}}} \right)^{0.1} \]
\[ \Psi_{sr} = \begin{cases} 1.35 \left( \frac{h_{ef}}{H} \right)^{0.25} \leq 1.20 & \text{for } H \leq 3.0 \cdot h_{ef} \\ 1.00 & \text{for } H > 3.0 \cdot h_{ef} \end{cases} \]

where \( H \): member thickness [mm], \( h_{ef} \): anchor embedment depth [mm], \( A_b \): anchor bearing area [mm²], and \( A_b^{\text{code}} \): a code-equivalent bearing area corresponding to a bearing pressure of \( \sigma_b=15 \cdot f_c \) under the anchor head at peak load which can be determined from (Eq. (5)).

\[ A_b^{\text{code}} = \frac{N_{\text{CC method}}}{15 \cdot f_c} = \frac{16.8 \sqrt{f_c(h_{ef})^{1.5}}}{15 \cdot f_c} \]

The detailed evaluations of the proposed modification factors are given Nilforoush et al. \(^{10,11}\). Based on the numerical results of headed anchors in concrete members of various thicknesses \( \Psi_H \) is limited to 1.20. The numerical results revealed that, for \( \Psi_H \leq 1.0 \), unreinforced concrete members fail by concrete splitting, whereas for \( \Psi_H \geq 1.0 \) both unreinforced and reinforced members fail via concrete cone breakout. The proposed \( \Psi_{sr} \) factor is applicable if concrete member is orthogonally reinforced and has a reinforcement-content of at least \( \rho=0.3\% \) in each direction. The factor \( \Psi_{sr} \) should not be used to calculate the concrete cone failure load in cracked concrete (Nilforoush et al. \(^{11}\)).
Note that headed studs are standardized and therefore when used as anchor with a small embedment depth the pressure under the head is much smaller than $p_h = 15 \cdot f_c$ (see Fig. 1b) resulting in an increase of the failure load in most cases to the value according to Eq. (1) (see Fig. 1a).

The proposed equation (3) should be used for the maximum embedment depths given in ACI 349$^3$ and ACI 318$^4$ (i.e. $h_{ef} \leq 635$ mm) only. To extend the application of equation (3) to headed anchors with $h_{ef} > 635$ mm, further numerical and experimental evaluations of anchor bolts at larger embedment depths (than considered in this study) are required. Further systematic experimental and numerical studies are also required to validate and generalize the proposed recommendations for anchor groups or anchor bolts placed close to concrete free edges.

7 Conclusions

In the present paper, the influence of member thickness, anchor head size, and orthogonal surface reinforcement in concrete walls and slabs on the tensile breakout capacity of single cast-in-place headed anchors embedded in uncracked concrete was evaluated both numerically and experimentally.

Based on the experimental and numerical results it was found that the tensile breakout capacity of anchor bolts in uncracked concrete increases up to 20% with increasing member thickness or if a light amount of orthogonal surface reinforcement ($\rho = 0.3\%$) is present at the anchoring zone. This is due to an increase of the global bending stiffness of the concrete member which causes an increase of the anchorage capacity and residual strength.

The anchorage fails by concrete splitting in relatively thin unreinforced concrete ($H < 2.0h_{ef}$). However, a light amount of orthogonal surface reinforcement can prevent concrete splitting failure and change the failure mode to concrete cone breakout, thereby increasing the anchorage capacity. It was also found that the increase rate due to surface reinforcement is dependent on the member thickness: the thinner the concrete member the larger the favorable influence of surface reinforcement on the anchorage capacity.

Furthermore, the tensile breakout capacity of anchor bolts increases by increasing the size of the anchor head. However, the anchorage behavior becomes stiffer and more brittle when enlarging the head size. The average concrete cone angle with respect to the loading direction increases with increasing head size. In the case of anchors with a large head, the diameter of the concrete cone at the surface of the member is $> 4.0h_{ef}$. This differs from the projected failure area $(3.0h_{ef} \times 3.0h_{ef})$ assumed by the CC method, and thus may affect the characteristic anchor spacing ($s_{cr}$) and edge distance ($c_{cr}$) of anchors with a large head.

Finally, it was found that over-reinforced concrete does not improve the anchorage capacity and performance any further than lightly-reinforced concrete.

To refine the CC method and better predict the tensile breakout capacity of cast-in-place headed anchors with various head sizes in unreinforced and reinforced uncracked concrete members of various thicknesses, the CC method (Eq. (1)) was modified and further extended as Eq. (3) by incorporating three modification factors to take the influence of member thickness, anchor head size,
and orthogonal surface reinforcement into account. The proposed equation (3) should be used for the maximum embedment depths given in ACI 349 and ACI 318 (i.e. $h_{ef} \leq 635$ mm) only. To extend its application to anchors with $h_{ef} > 635$ mm, further numerical and experimental investigations with anchor bolts at larger embedment depths than those considered in this study are required.

8 Acknowledgement

The authors acknowledge the financial support by Energiforsk, a Swedish Energy Research Centre, for this research project.

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BOND BETWEEN REINFORCEMENT AND CONCRETE
EXPERIMENTAL INVESTIGATION OF THE LOCAL STRAIN FIELD IN A RC PULL-OUT TEST

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ABSTRACT

RC is based on the fundamental feature of bond between the rebar and the concrete. Studies on bond are commonly done through pull out tests, where the applied load and the relative pull-out displacement (slip) are recorded.

This paper aims at presenting early results regarding the local interaction between the rebar ribs and the concrete using an innovative experimental procedure. It aims at gaining insight and better understanding of the mechanisms governing the rebar-concrete bond at the local level. The new experimental set-up enables direct continuous observation and measurements of the concrete displacements and yields the continuous displacements and strain fields near the rebar rib, with images decoding technique based on image correlation. The set-up is based on a standard pull-out test of a cylindrical specimen in which a 90° “window” sector is left open during its casting. Measurements of the concrete displacements are performed by a sequence of timed high-resolution digital photographs that are later analyzed by image correlation. A very good match is obtained between the external LVDT measured slips and those obtained from the image analysis. Comparisons have been made between the load-displacement relationships obtained in the "window" specimens and control specimens (without a "window") and a minor effect of the "window" on part of the global response is observed. This new procedure enables a continuous recording of the evolving displacements and strains in concrete near the ribs, as well as follow up of the developing local damage. Such experimental observations that were not available until now may enhance the understanding and advanced modelling of bond mechanics.

1 Introduction

The key feature that enables the structural composite behavior of reinforcing concrete is the rebar-concrete bond. Deformed rebars ribs develop mechanical interaction that enables load transfer between the rebar and the surrounding concrete. It is commonly assumed that the "bond tractions" that develop at the steel-concrete interface are uniformly distributed along the contact area. Study of bond is commonly carried out through pull-out test of rebar that are centrally embedded in prismatic or cylindrical concrete specimens. External monitoring of the applied load and slip (relative displacement between the rebar and the concrete) enables to experimentally define "bond-slip relationships" which are implemented in advanced analyses. These common experimental
procedures provide an experimental black box with no information on the interior local interaction between the rebar and the surrounding concrete, and cannot provide any measurements of concrete displacement, strains or any local damage characteristics in the concrete vicinity to the rebar. Several studies focused on the cracking development\textsuperscript{6,7} but the cracks were examined only at the end of the test, or through a relative new technique based on X-Ray\textsuperscript{8}, at several distinct stages. While vast knowledge exists regarding the global bond behavior, there is almost no information on the local bond mechanisms. Improvement of our understanding of these mechanisms is the aim of the new experimental procedure. Continuous observation and measurements of the concrete displacements near the ribs during the pull-out test, make it possible to achieve that goal.

2 Experimental Setup

2.1 General

The proposed experimental setup is based on a standard pull-out test\textsuperscript{1} of a cylinder concrete specimen with a centrally embedded rebar. The setup is shown in Fig. 1. The applied load, the rebar-concrete relative displacement (slip) and the deformations in the concrete in the vicinity of the rebar's ribs are measured during the test.

![Figure 1: Experimental setup](image)

The specimen has a 90-degree void sector that is formed during casting to enable a direct close examination of the rebar concrete interface. In the new proposed technique, the "window" has two opposing aspects:

- The "window" locally disrupts the cylindrical symmetry around the rebar and reduces the rebar-concrete contact area by 25%.
- The "window" allows a continuous monitoring of the concrete displacements during pullout of the rebar.

The measurement of the concrete in the vicinity of the rebar is done by high-resolution digital photography of the concrete and is an important part of this setup. The window's height should allow suitable lightening of the photographed area on the one hand and it should be limited as much as possible in order to minimize its disturbance, on the other hand.

The records include the pull-out force, externally measured rebar relative displacement and consecutive photographs taken at a frequency which allows synchronization with the above-mentioned records of the test.
The experiments were performed using a Hydraulic MTS810-100kN loading frame. The rebar was pulled at a rate of 0.04 mm/min and the pictures of the photographed area were taken every two seconds. The external rebar displacement measurements include three LVDT (linear variable displacement transducers) that measured the top and bottom slip (see Fig. 1).

Full details of the technical requirements and verification of the deformations acquisition and analysis are described in an earlier recent article. For clarity, a summary of the technique is briefly given here. Correspondence between the location of a single point in one image and in the following image enables to derive the displacements fields in a set of consecutive pictures. Spraying small dots with a contrasting color from a distance of about 30 cm that are distributed randomly in location and size (see Fig 2) give a non-periodic surface which give the best analysis results (density of 5-10 dots per mm). Alignment of the camera perpendicular to the photographed area enables proper photography and analysis of the photographs. The calibration coefficient in the tests was 0.03-0.04 mm per pixel. This resolution matches an error value of 0.001-0.02 pixels, which is equal to $0.035 \times 10^{-4} - 7.00 \times 10^{-4}$ mm. Analysis of the photographs by image correlation analysis with VIC2D software enables to get the displacement and strain fields in the concrete near the ribs.

Figure 2: Sprayed concrete of the photographed area of interest

### 2.2 Height of the “Window”

The window's height should be minimized because opening of a "window" in the cylindrical specimen violates the axial symmetry, but the height should be sufficiently large to allow proper photography. From the photography considerations, it was found that the minimal distance that allows direct observation of the interaction processes between the rebar and the surrounding concrete is 40 mm. In order to examine the “window effect”, two "window" heights, 40 and 80 mm, were examined, both experimentally and numerically. The conclusion from a numerical sensitivity analysis, as well as results from preliminary tests, was that a relative small "window" height of 40 mm allows quality photography and provides a global bond behavior that is similar to that of a full specimen without a "window", up to the maximum pull-out load, while it slightly affects the post peak resistance along the descending branch.

Upon confirmation, the "window" concept, the following research has focused on the digital photography of the concrete near the ribs and on the image correlation analysis.
3 Experimental program

3.1 Specimens

Two types of specimens were examined:

- concrete cylinders including an observation “window”.
- full concrete cylindrical control specimens without a “window”.

Custom-made rebars were used in this study, which were prepared from 20-mm diameter steel rods. The rebars were bonded to the concrete along either two or five ribs (nominal bonded lengths of 30 and 75 mm). Plastic tubes isolated the un-bonded parts of the rebar from the surrounding concrete and the posttest actual bonded lengths were measured for each specimen (Table 1). In specimens with a "window" and two bonded ribs, the "window" provided a direct view at the ribs. In five-ribs specimens, where the “window” height is shorter than the bonded length, the “window” was located either in the region of the first two ribs (closer to the loaded end of the rebar) or at the region of the third and fourth ribs. Each specimen was denoted as ID-D-N-O (1, 2 or 3, 4), where:

ID - The casting group. Control specimens have an additional letter F as suffix (e.g. TOLF).
D - Specimen diameter (cm).
N - Number of ribs in the bonded length.
O - Serial number of the test series with the same parameters.

(1, 2 or 3, 4) - For specimens with five ribs – the location of the "window", exposing either ribs #1 and #2 or #3 and #4.

For example: specimen TOL-20-2-1 is the first specimen of series TOL; this is a 20-cm diameter cylinder with a window, with a 2-ribs rebar in contact with the concrete. Details of the test specimens are given in Table 1. The experimental program includes twelve specimens. This paper presents the early results of 5 tests (marked in bold in Table 1).

Table 1: Specimen details

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete strength (MPa)</th>
<th>Specimen diameter (cm)</th>
<th>Window's height (cm)</th>
<th>Number of ribs</th>
<th>Actual Bonded length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SO-10-2-2</td>
<td>33.6</td>
<td>10</td>
<td>4</td>
<td>2</td>
<td>28</td>
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<tr>
<td>SO-20-2-1</td>
<td>33.6</td>
<td>20</td>
<td>4</td>
<td>2</td>
<td>28</td>
</tr>
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<td>TOL-10-2-1</td>
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<td>10</td>
<td>4</td>
<td>2</td>
<td>28</td>
</tr>
<tr>
<td>TOL-20-2-1</td>
<td>43.0</td>
<td>20</td>
<td>4</td>
<td>2</td>
<td>37</td>
</tr>
<tr>
<td>TOL-20-5-1(1,2)</td>
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<td>20</td>
<td>4</td>
<td>5</td>
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</tr>
<tr>
<td>TOL-20-5-2(3,4)</td>
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<td>20</td>
<td>4</td>
<td>5</td>
<td>79</td>
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<tr>
<td>SOF-10-2-1</td>
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<td>10</td>
<td>--</td>
<td>2</td>
<td>26</td>
</tr>
<tr>
<td>TOLF-10-2-1</td>
<td>43.0</td>
<td>10</td>
<td>--</td>
<td>2</td>
<td>34</td>
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<td>TOLF-10-2-2</td>
<td>43.0</td>
<td>10</td>
<td>--</td>
<td>2</td>
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<td>--</td>
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<td>--</td>
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<td>--</td>
<td>5</td>
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</tr>
<tr>
<td>TOLF-20-5-2</td>
<td>43.0</td>
<td>20</td>
<td>--</td>
<td>5</td>
<td>86</td>
</tr>
</tbody>
</table>

{1} Control specimen (without a “window”); {2} Measured at 28 days on 10-cm cubes (7-days wet cured).
3.2 Materials

Concrete

In order to minimize heterogeneity of the different specimens, the concrete mixtures included 1354 kg/m³ (and 1233 kg/m³ in the second mix) natural sand without coarser aggregates, 500 kg/m³ cement (CEM I-52.5N taken from two different batches in the two different castings that were made several months apart), and 300 l/m³ water. These mixtures were prepared with somewhat different ingredients and different vibration periods, which produced two different concrete strengths of 33.6MPa and 43.0MPa, as reported in Table 1.

Rebar

Custom-made 20-mm diameter rebars with engraved ribs were prepared (Figure 3). The ribs of the tailored bars had a uniform cross-section and they were perpendicular to the bar’s longitudinal axis. These bars were manufactured for the current series of tests in order to eliminate effects that may be associated with the varying shape of the rib cross-section, as well as their inclination, that are typical to standard rebars. The tailored bars were prepared from a smooth circular Fe360 grade rods with a rib spacing of 15 mm. The height of these ribs was 1.5 mm and their face angle was 59°. The height and spacing of the ribs correspond to a 'relative rib area', \( f_r = 0.11 \) (where \( f_r = \frac{\text{projected rib area normal to bar axis}}{\text{(nominal bar perimeter) \cdot (center – to – center rib spacing)}} \)) which is very close to that of a standard rebar.

4 Test results

4.1 Average bond stress -slip curves

The average bond stress is commonly assumed as evenly distributed along the bonded rebar. It is calculated by dividing the pull-out force by the bonded area. Figures 4a, and 4c show comparisons of the average bond stress (calculated according to the actual bonded area as mentioned above; refer to Table 1) vs. the measured relative displacement (slip) for full and "window" specimens with the same conditions (i.e. same diameter, type of concrete and number of ribs). The comparison shows that in 200-mm diameter specimen with two bonded ribs, the maximum average bond stress of the two specimens (full and with "window") were almost the same (Fig. 4a). However, in 100-mm diameter specimens, where less confinement was provided, the control (full) specimen reached a somewhat higher peak bond stress (by about 13%) but at the same displacement (slip). Differences
between the full and “window” specimens were mainly observed in the descending branch. Figures 4b and 4d, show the normalized bond-slip curves (actual displacement normalized with respect to the displacement obtained at the peak average bond stress and average bond stress normalized with respect to its maximum value). The normalized curves of both the control specimen and of the "window" specimen behave similarly along the ascending branch and beyond, along part of the descending branch. This similar behavior gives an indication of the development of similar internal mechanisms during the developing bond resistance up to the peak load.

![Normalized bond-slip curves](image)

Figure 4: Absolute (a, c) and normalized (b, d) average bond stress-slip curves of full and “window” specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>diameter (cm)</th>
<th>Number of ribs</th>
<th>Max. load (kN)</th>
<th>Max. bond stress (MPa)</th>
<th>Slip at max. load (mm)</th>
<th>Actual bonded length (mm)</th>
</tr>
</thead>
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<tr>
<td>SO-10-2-2</td>
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<td>2</td>
<td>9.32</td>
<td>9.54</td>
<td>0.94</td>
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<td>SO-20-2-1</td>
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<td>21.88</td>
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<td>TOL-20-5-1</td>
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<td>75</td>
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<td>TOL-20-5-2</td>
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<td>54.85</td>
<td>13.63</td>
<td>0.6</td>
<td>79</td>
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<td>SOF-10-2-1 [1]</td>
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<td>1.03</td>
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<td>32.29</td>
<td>14.62</td>
<td>1.8</td>
<td>38</td>
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<tr>
<td>SOF-20-2-1 [1]</td>
<td>20</td>
<td>2</td>
<td>20.32</td>
<td>12.95</td>
<td>1.15</td>
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</tr>
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<td>TOLF-20-5-1 [1]</td>
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<td>76.45</td>
<td>16.87</td>
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<td>83.24</td>
<td>16.65</td>
<td>1.33</td>
<td>86</td>
</tr>
</tbody>
</table>

[1] Control specimen (without a window);
4.2 Elimination of Rigid Body Displacement.

Verification of the accuracy of the image analysis procedure was done by its comparison with the conventionally measured relative displacement (i.e., measured by the LVDT). In order to compare the LVDT measurements with the image analysis results, rigid body displacement (RBD) had to be eliminated from the latter. The RBD includes all the displacement components, such as compression of the Teflon layer that was placed under the specimen, deflections of the top frame beams and deformation of the rods hanging from them (see Fig. 1), possible displacements due to the hydraulic system etc. RBD were obtained by analyzing the photographed dots sprayed on the unstressed outer plastic tube (see Fig. 2). Fig. 5 shows load-displacement curves of the pulled-out rebar, which were obtained using the external LVDT. It also shows the results obtained from the analyzed images of the rebar displacement – before and after the RBD elimination. Typical results are shown for a 100 and 200-mm diameter two-ribs specimens. It is evident from Fig. 5 that elimination of the RBD from the displacements obtained from the image analysis ("VIC2D" in Fig. 5) yields net relative displacements that agree very well with the measured results obtained from the external LVDT.

![Image](a) SO-10-2-2  (b) TOL-20-2-1

Figure 5: Elimination of RBD from the rebar load-displacement curve for two bonded ribs.

After this discussion of the bond-slip behavior that focused on the rebar relative displacement during the pull-out test, the following sections will give attention to the local behavior of the concrete surrounding the rebar.

4.3 Image analysis of the displacement fields

The image correlation analysis uses the VIC2D\textsuperscript{10} software to process the recorded data. Figure 6 shows an example of the longitudinal displacement field, in the direction of the pull-out load, for several load levels (denoted as a fraction of the peak load), where the horizontal axis is parallel to the rebar axis (i.e. longitudinal direction), and the vertical axis is in the radial direction. The colored plots show the magnitude of the longitudinal displacement. Figure 6 shows that the longitudinal photograph-processed displacements at the ribs base for all load levels are compatible with the measured LVDT displacements. For example, the displacement measured by VIC2D\textsuperscript{10} was 0.063 mm and the displacement measured near the first rib (where "first" denotes the rib closer to the point of load application) was 0.060 mm (Fig. 6a). Figure 6c shows the diagonal crack development, which created a concrete region that is displaced with the rebar. Figure. 6d shows the longitudinal displacement filed for two bonded ribs; we can see that near the second rib no diagonal crack is developed and the displacement at the base of the second rib and in the concrete outside the diagonal
crack at the first rib are alike (azure color). Figs. 6e and 6f show the longitudinal displacement in specimen TOL-20-5-1 (five bonded ribs where the "window" was placed at the location of the first and second ribs). At a low load level (Fig.6e) both ribs develop similar displacement fields at their vicinity, but at an increasing load (Fig. 6f) an extended displacement field is observed in front of the first rib although in the base of both ribs the displacement is identical.

(a) $F/F_{\text{max}}=0.19$ Measured displacement by LVDT =0.063mm (Specimen TOL-10-2-1 first rib)

(b) $F/F_{\text{max}}=0.4$ Measured displacement by LVDT =0.19mm (Specimen TOL-10-2-1 first rib)

(c) $F/F_{\text{max}}=0.68$ Measured displacement by LVDT =0.31mm (Specimen TOL-10-2-1 first rib)

(d) $F/F_{\text{max}}=0.68$ Measured displacement by LVDT =0.31mm (Specimen TOL-10-2-1)

(e) $F/F_{\text{max}}=0.37$ Measured displacement by LVDT =0.1mm (Specimen TOL-20-5-1)

(f) $F/F_{\text{max}}=0.74$ Measured displacement by LVDT =0.35mm (Specimen TOL-20-5-1)

Figure 6: Longitudinal displacement field at different load levels

Radial displacement fields of specimen TOL-20-5-1 are illustrated in Figure 7. At a low load level, the radial displacement fields observed in the vicinity of the two bonded ribs are somewhat similar, but as the load increases an uneven distribution is observed.
4.4 Image analysis of the strain fields

The strain fields are obtained from the displacement fields by calculating the Lagrangian strains from the following displacement gradient equation:

\[
E_{ij} = \frac{1}{2} \left( \frac{\partial d_i}{\partial X_j} + \frac{\partial d_j}{\partial X_i} + \sum_{k=1}^{2} \left( \frac{\partial d_k}{\partial X_j} \frac{\partial d_k}{\partial X_i} \right) \right),
\]

where \( d_i, d_j \) are the displacement vectors and \( X_i, X_j \) are coordinates of the deformed position of a general point. It is important to note that the values of strain in the vicinity of cracks include the effect of the crack opening.

Figure 8 shows the development of the maximum principal strains near the first rib of specimen TOL-10-2-1. A gradual evolution of the diagonal crack at the rib edge is observed. It should be noted that with increase of the displacement concrete in the bases of the ribs was damaged and crushed and from that stage the local strains in the ribs vicinity could not be further analyzed.
Analysis of the shear strain fields shows that a relatively small size zone of large shear strains is developed in the rebar vicinity (Fig. 9a). As the pullout load increases the width of the large shear strain zone and the strain magnitude increases as well and it develops firstly near the first rib (Fig. 9b). Numerical examination of the shear strain fields enable to identify a limit beyond which the shear strain is almost zero. This limit is marked with a dashed white line in Fig. 9b.

Figure 9: Shear strain fields at different load levels in specimen SO-20-2-1.

5 Summary and Conclusion

This article describes new experimental findings that were obtained from a new experimental procedure that allows continuous information of data. The image correlation analysis of the photographs allows measuring the longitudinal and radial displacements fields in the concrete near the ribs for the first time. The obtained data show the following findings:

- A high displacement gradients zone is developed in front of the rib. This zone is expanding with increase of the pull-out load.
- At a low load level, similar radial displacement fields are observed at the two bonded ribs areas, but as the load increases, the radial displacements become uneven.
- The maximum principal strain fields (derived from the displacements), indicate the early development of an inclined crack and is capable of following its growth.
- Analysis of the shear strain fields show that a relatively limited zone of large shear strain is developed close to the rebar. The width of the zone and the strain magnitude increase with increase of the pullout load.

The digital image analysis provides continuous numerical information of the displacement fields in the concrete near the ribs at every load level during a pull-out test. This is a unique resource of new data that sheds light on rebar-concrete interaction.

6 Acknowledgement

This work is partly supported by the Israeli Ministry of Science, Technology and Space. The research grant is greatly appreciated. The authors would like to thank also E. Itzhak and E. Gershengorn for their valuable technical support.
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THE RESEARCH ON BONDING BETWEEN CONCRETE AND REINFORCEMENT

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ABSTRACT

The condition due to a good bond between reinforcement and concrete is provided that bar is not pulled or pushed out from concrete and they can work as a solid body and together will receive force. The bonding behavior mainly depends on profile of reinforcing bar, friction force on interaction surface of bar and concrete, cement colloid quality and splices and anchor length of bar. The purpose of the research is to check bonding capacity between concrete and reinforcing bar for splices of reinforcing bar.

Valuable experience applying standard and Mongolian laboratory testing procedures, especially for lap splices of bar in beam, data analysis and developing relationships between concrete and reinforcement was obtained. Three kind of beams were made for the study. The first beam is with a continuous reinforcement bar, the second beam is with horizontally overlapped bottom reinforcement bars and the third one is with vertically overlapped bottom reinforcement bars. Strain of bar, bearing capacity and deflection of beam and crack were measured.

1 Introduction

Reinforcing bars delivered at site are required to be connected and spliced due to insufficient length, especially when construction structures have large span. Wire of less than 8 mm diameter, B500 grade is produced and delivered in packing of circle for our user. But bars of greater than 10 mm diameter are delivered as 6.5 m to 14 m length for user, so there is a need to connect and splice them in some cases. There are two basic categories of splices:

-Weld or mechanical splice
-Lap splice /no welding/

Weld or mechanical splicing is carried out in accordance with the MNS ISO 15835-2016 and MNS 6388:2013 standards.

The second category depends on reinforcing bar type and structural solution. The aim of the research is to study the bonding capacity between concrete and reinforcing bar for lap splices of reinforcing bars. It is well known that overall behavior of reinforced concrete structures is highly dependent on interaction between steel and concrete. The interaction between concrete and reinforcing bar is due...
to, chemical adhesion, friction, mechanical interlock and shear along cylindrical concrete surface between adjacent ribs (Belarbi et al. 2010)\textsuperscript{9,10}.

The lap length of reinforcing bars in tension and compression should be greater than the value of the following formula.

\[
\ell_i = \alpha \ell_{0,an} \frac{A_{s,cal}}{A_{s,ef}}
\]

\(\ell_{0,an}\) - basic length of embedment;

\(A_{s,cal}, A_{s,ef}\) - cross section area corresponding calculation and effective of reinforcing bar, respectively.

\(\alpha\) - factor of influencing in embedment length depending on stress state between concrete and bar and structural solution around anchorage zone. Whether the effective lap length of reinforcing bar should be no less than \(0.4\alpha \ell_{0,an}\) or \(20d_s\) or /and 250 mm.

Bond strength for interaction between concrete and steel bar is obtained by the following formula.

\[
\tau_m = \frac{N}{l_{an} \cdot U}
\]

Here;

\(l_{an}\) – embedment length of bar, \(U = \pi \cdot d_s\) – perimeter of reinforcing bar, \(N\) – axial load of bar in concrete.

The bond between concrete and bar is studied for beams with continuous reinforcement bars, with horizontally overlapped bottom reinforcement bar and with vertically overlapped bottom reinforcement bars.

2 Experimental Procedure

Material used in the test: Concrete - strength (FCK = 45MPa by cubic mold)

Reinforcing bar - Diameter of reinforcement bar is 25 mm (Longitudinal Bar) 13 mm (Stirrup Bar) and lapping length is 850 mm.

Three beams of 6.5 m length, 0.4 m width and 0.4 m height were prepared for the test as shown in Figure 1. The first beam is with continuous reinforcement bar, the second beam is with horizontally overlapped bottom reinforcement bars and the third one is with vertically overlapped bottom reinforced bars as shown in Figures 2 to 4.

Equipment used in the test consists of data logger (TDS-102), strain gauge (FLA-5-11-5L), pressing jack, meter, deflection meter, etc.

The beam is loaded with jack in the steps of one ton until ultimate load. Strain of bar, bearing capacity and deflection of beam and crack were measured.
Figure 1: Beam elevation and section A-A

Figure 2: Section and strain gauge position for the 1st beam with continuous bar

Figure 3: Section and strain gauge position for the 2nd beam with horizontally overlapped bottom reinforcement bar
1.1 Result and Discussion

A Comparison of deflection for three beams is shown as figure 5. The Deflection of the 3\textsuperscript{rd} beam is slightly higher than of the 1\textsuperscript{st} and the 2\textsuperscript{nd} beams. As for the 1\textsuperscript{st} beam with continuous bar, deflection is lower than others. Deflection of the 1\textsuperscript{st}, 2\textsuperscript{nd} and 3\textsuperscript{rd} beam were about 3.5 cm, 3.4 cm and 4.1 cm at 28 ton of load, respectively. Load bearing capacity of the beam according to the calculation is about 28 t. In this case, three beams deflection is 10-24\% higher than analytic value at 28 ton load.

As shown in Figure 6 to 8, strain of bar is rapidly increasing at 28 ton of load.
3 Analysis

Receiving force of bar and bonding average stress at 28 t load is calculated in Table 1.

Table 1: Stress comparison of bar in beams

<table>
<thead>
<tr>
<th>1st beam</th>
<th>2nd beam</th>
<th>3rd beam</th>
<th>Force receiving</th>
<th>Force receiving</th>
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<tbody>
<tr>
<td>$\varepsilon$</td>
<td>$N_s$</td>
<td>$\tau_m$</td>
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<td>$N_s$</td>
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<td>1983</td>
<td>9.73E+05</td>
<td>2.92E+06</td>
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<td>9.49E+05</td>
</tr>
</tbody>
</table>

Average: 1.45E+06 4.35E+06 8.09E+05 2.42E+06 55.75 1615 7.92E+05 2.37E+06 54.60

Receiving force of bar is calculated as following formula.

$$N_s = R_s A_s = \varepsilon_s E_s A_s \leq R_s A_s$$  \hspace{1cm} (1)

Bonding average stress is given by

$$\tau_m = \frac{\sigma_s \cdot d_s}{4 \ell_{an}} = \frac{\varepsilon_s \cdot E_s \cdot d_s}{4 \ell_{an}}$$  \hspace{1cm} (2)

Bonding average stress of the bars in the “2nd” and “3rd” beams is compared with bonding average stress of the bars in the “1st” beam. The percentage of force taken up by reinforcing bars in the 2nd and “3rd” beam is 55.8% and 54.65% of the “1st” beam, separately.
The first and the third beam were analyzed on LIRA program, because the deflection of the third beam is higher than of the second beam.

Figure 9: 1st beam for LIRA program

Figure 10: 3rd beam for LIRA program

Figure 11: Deflection at 28 t load of 1st beam
The first and the second beam were analyzed on LIRA program. Result of deflection and stress were different than our test results by 25-28 percent.
4 Conclusion

1. It is sufficient that the bonding capacity of the overlapped reinforcement bar is 54.6-55.75% of the capacity of the continuous reinforcement bar.
2. The bonding capacity of the overlapped bottom reinforcement bar/vertical side for 3rd beam is smaller of 1.2% than the bonding capacity of the bar in the 2nd beam.
3. As seen from ultimate load, maximum load for the 3rd beam and the other two beams is 28 t and 30 t separately. It is sufficient to compare the results of the calculation.
4. Deflection of the 3rd beam is larger than of the other beams. Therefore, the deflection of the three beams is higher by 10-24% than allowable deflection under more than 24 t load.

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ANCHORAGE OF LARGE-DIAMETER REINFORCING BARS

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ABSTRACT

The use of large-diameter reinforcing bars $> 32$ mm in structural concrete elements designed according to Eurocode 2 is subject to additional rules. Additional rules for anchorage zones were derived from technical approvals, while systematic tests on anchorages of large diameter bars have not yet been conducted. Beam-end tests and four-point bending tests on lapped splices with large diameter bars showed that the transition from conventional to large reinforcing bars is fluent and that some additional rules may be reduced. Therefore, tests on anchorages of large diameter bars are conducted at RWTH Aachen University. The test parameters are chosen according to the additional rules in Eurocode 2. For higher splitting forces and greater dowel action, transverse reinforcement is required for large diameter bars. Eurocode 2 specifies two types of transverse reinforcement for the use of large diameter bars: surface reinforcement for crack control and links as confining reinforcement for anchorage zones where transverse compression is not present. Another confining reinforcement along the full length of the structural member, which is specified in the German National Annex, provides a third type of transverse reinforcement for the use of large diameter bars. By the anchorage tests, the effectively needed confinement in anchorages with large diameter bars and high reinforcement ratios will be investigated. The conference paper presents the anchorage-test parameter, the test setup and the deformation measurements during the tests. Test results will be evaluated with regard to the necessity of additional rules for large diameter bars in anchorages.

1 Introduction

Eurocode 2\textsuperscript{1} contains various additional rules for large diameter bars. The definition of “large diameter” is a nationally determined parameter. The recommended value of $\Omega_{\text{large}} = 40$ mm given in Eurocode 2\textsuperscript{1} was adopted in most European countries. The high rebar force developed by large-diameter reinforcing bars leads to considerable hoop stress around the bars. Since the bond strength does not increase for large diameter bars, splitting is more likely to occur. To prevent brittle bond failure, Eurocode 2\textsuperscript{1} and the German National Annex\textsuperscript{2} include many additional rules for the use of large diameter bars.

The necessity of the main additional rules were investigated in a collaborative research project funded by the Programme for Sponsorship by Industrial Joint Research and Development (AiF, IGF number 16992 N/1)\textsuperscript{3}. Within the project, the bond behaviour, crack control and columns with large diameter bars were analysed. The bond-behaviour investigation comprised beam-end tests and laps under tension. Since many additional rules regard the anchorage of large diameter bars,
anchorages at beam ends are currently investigated within a collaborative follow-up project (AiF, IGF number 18821 N/1).

In most research projects, the bond behaviour was investigated by pull-out or beam-end tests\textsuperscript{4,5,6}. These test specimens were usually conducted with short bond lengths and the bond strength was therefore assumed to be constant. For the short bond length, the reinforcing bar slips both at the loaded and the unloaded side of the bond length. This behaviour is contrary to real anchorages with longer bond length and zero slip at the unloaded side of the reinforcing bar. Anchorages mainly occur at beam ends, in zones of curtailed reinforcement and in foundations (Figure 1-1).

![Anchorages at beam ends and curtailed reinforcement](image)

Figure 1-1: Anchorages at beam ends and curtailed reinforcement

Since conventional bond tests differ from anchorages in building practice, single-span beam tests as documented by Richter\textsuperscript{7}, Andreasen\textsuperscript{8} and Tahershami et al.\textsuperscript{9} are necessary to analyse the anchorage bearing capacity. The realisation of anchorages at beam ends is particularly difficult for large diameter bars (\(\Phi > 32\) mm), since hooks and bends are hardly feasible. Therefore, twelve anchorages with straight reinforcing bars with large diameters and high reinforcing ratios are tested within the current research project.

The required bond length at beam ends has to be designed to transfer the force resulting from the reinforcing stress at the front edge of the support. This stress is calculated from the additional tensile force derived from the shear model, while the bending stress is almost zero at the support. In practice, the anchorage length begins at the last bending crack before the support. So while the designer regards the anchorage length beginning at the front edge of the support, the real anchorage length commences in the shear span and is thus considerably longer (Figure 1-2). Besides the anchorage length, also the transverse pressure differs. While a beam end is subjected to considerable transverse pressure at the support, the pure bending zone in front of the support is not subjected to transverse pressure.

![Bond length with and without induced crack](image)

Figure 1-2: bond length with and without induced crack

For anchorage testing, a crack can be induced at the front edge of the support to adjust the test to the basis of anchorage design. This approach leads to a defined anchorage length. Alternatively, the real
bending behaviour is accepted. Mari\textsuperscript{10} introduced a design approach for the determination of the position of the last bending crack before the support. This approach leads to a defined anchorage length as well, however partly with and without transverse pressure. Moreover, the position of the last bending crack does not only depend on the test dimensions but also on the random distribution of tensile concrete strength along the beam. Therefore, induced cracks were implemented for the test specimens conducted at RWTH Aachen University.

### 2 Confining transverse reinforcement for large diameter bars

Eurocode 2\textsuperscript{1} contains four rules for confining reinforcement for bond and the German National Annex\textsuperscript{2} adds a fifth rule. There are two rules for anchorages and laps for all bar diameters (\(\Sigma A_{st} \geq 0.25 \cdot A_s\) for anchorages, \(\Sigma A_{st} \geq 1.0 \cdot A_s\) for laps) and one rule explicitly for anchorages of large diameter bars (\(A_{sv}, A_{sh},\) Figure 2-1). The surface reinforcement for crack control for large diameter bars also includes confining reinforcement (\(A_{s,surf} \geq 0.01 \cdot A_{ct,ext},\) Figure 2-2). The German National Annex\textsuperscript{2} adds another rule for confining reinforcement for large diameter bars (0.1 \(\cdot A_s\)). The aim of the conducted research on laps and anchorages with large diameter bars and high reinforcement ratios is to verify the necessity and to reduce the number of confining reinforcement rules.

\textbf{Figure 2-1: Transverse reinforcement in anchorages without transverse pressure}

\textbf{Figure 2-2: Surface Reinforcement}

<table>
<thead>
<tr>
<th>Eurocode 2 requirements for confining reinforcement in bond zones</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Anchorage</strong></td>
</tr>
<tr>
<td>8 mm - 32 mm</td>
</tr>
<tr>
<td>(\Sigma A_{st} / A_s \geq 0.25) (beams)</td>
</tr>
<tr>
<td>(\Sigma A_{st} / A_s \geq 0) (slabs)</td>
</tr>
<tr>
<td>(\Sigma A_{sh} / A_s \geq 0.25)\textsuperscript{1}</td>
</tr>
<tr>
<td>NAD (A_{it} \geq 0.1\Sigma A_s) [cm(^2)/m]</td>
</tr>
<tr>
<td>Crack control by (A_{s,surf}: A_{st} \geq 0.01\cdot A_{ct,ext}) \textsuperscript{1}</td>
</tr>
</tbody>
</table>

\textsuperscript{1} For one reinforcement layer
Where

- \( A_s \) area of one anchored or lapped bar
- \( \Sigma A_s \) are of the sum of all bars anchored or lapped at a section
- \( A_{s,\text{surf}} \) area of the surface reinforcement
- \( A_{\text{ct,ext}} \) area of the tensile concrete external to the links
- \( A_{st} \) area of the transverse reinforcement
- \( \Sigma A_{st} \) sum of the area of the transverse reinforcement along the anchorage or lap length
- \( \Sigma A_{sh} \) horizontal transverse reinforcement crossing the vertical plane of splitting
- \( \Sigma A_{sv} \) vertical transverse reinforcement crossing the horizontal plane of splitting

The reinforcement required to secure adequate bond according to the German National Annex\(^2\) equals 0.1 \( A_s \) along the structural element. One transverse reinforcement element may embrace up to three longitudinal bars and the distance may to be not less than 200 mm.

### 3 Previous experimental comparison of confining reinforcement for large diameter bars

Within the first research project\(^3\), the validity of the bond-strength reduction factor \( \eta_2 \) and the \( \alpha \) factors considering the effects of concrete cover and transverse pressure were confirmed for large diameter bars. Three-point bending tests with lapped splices in the constant moment zone and beam-end tests were tested to investigate the effects of lap length, bar diameter, concrete cover, transverse reinforcement and surface reinforcement.

The sum of the area of the transverse reinforcement positioned in the outer sections of the laps corresponded with the area of one lapped bar. In test specimen T6 with a lap factor \( \alpha_6 = 2.0 \) and a lap length \( l_0 = 44 \cdot \phi \) respectively, the transverse reinforcement was increased by reducing the stirrup distance. This modification did not lead to a considerable increase in lap-bearing capacity.

Several lap tests were conducted with surface reinforcement for crack control at the lap end. Test specimens with \( \alpha_6 = 1.5 \), transverse lap reinforcement according to Eurocode 2\(^1\) and surface reinforcement were able to develop longitudinal-reinforcement stress well above the yield strength (e.g. T9). Test T1 with transverse lap reinforcement only and test T11 with surface reinforcement only both failed at about 90 % of the bending moment calculated with the yield strength \( f_{ys} \) (Table 3-1).

<table>
<thead>
<tr>
<th>Test</th>
<th>Bar-Ø [mm]</th>
<th>Lap length ( l_0 )</th>
<th>Concrete cover ( c )</th>
<th>Bar distance</th>
<th>Transverse reinforcement ( A_{st} )</th>
<th>( \Sigma A_{st}/A_s )</th>
<th>( A_{s,\text{surf}} )</th>
<th>( M_{\text{test}}/M_{fy} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>40</td>
<td>33 \cdot \phi</td>
<td>1.5 \cdot \phi</td>
<td>2.0 \cdot \phi</td>
<td>\ Ø 14 / 13</td>
<td>1.0</td>
<td>-</td>
<td>89%</td>
</tr>
<tr>
<td>T9</td>
<td>40</td>
<td>33 \cdot \phi</td>
<td>1.5 \cdot \phi</td>
<td>2.0 \cdot \phi</td>
<td>\ Ø 14 / 13</td>
<td>1.0</td>
<td>1% ( A_{ct,ext} )</td>
<td>109%</td>
</tr>
<tr>
<td>T11</td>
<td>40</td>
<td>33 \cdot \phi</td>
<td>1.5 \cdot \phi</td>
<td>2.0 \cdot \phi</td>
<td>-</td>
<td>0.0</td>
<td>1% ( A_{ct,ext} )</td>
<td>91%</td>
</tr>
<tr>
<td>T3</td>
<td>40</td>
<td>44 \cdot \phi</td>
<td>1.5 \cdot \phi</td>
<td>2.0 \cdot \phi</td>
<td>\ Ø 14 / 13</td>
<td>1.0</td>
<td>-</td>
<td>100%</td>
</tr>
<tr>
<td>T6</td>
<td>40</td>
<td>44 \cdot \phi</td>
<td>1.5 \cdot \phi</td>
<td>2.0 \cdot \phi</td>
<td>\ Ø 14 / 9</td>
<td>1.7</td>
<td>-</td>
<td>102%</td>
</tr>
</tbody>
</table>
The conducted beam-end tests BEV1 reached a maximum normalised bond strength $\tau_{\text{max,norm}}$ of 10 N/mm² when two 14 mm stirrups were positioned in the bond length (corresponding with $\Sigma A_s = 1.0$ along the lap length). The mean bond strength of the beam-end tests BEV14 without transverse reinforcement in the bond zone was 8.7 N/mm². When positioning stirrups with the same distance but with smaller bar diameters in BEV8, the maximum normalised bond strength was reduced to 8.8 N/mm². The beam-end tests BEV19 with smaller bar diameters and smaller bar distance (corresponding to $\Sigma A_s = 1.0$ along the lap length as well) also reached $\tau_{\text{max,norm}} = 10$ N/mm² (Table 3-2).

Table 3-2: Influence of transverse reinforcement in beam-end tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Bar-Ø [mm]</th>
<th>Bond length $l_b$</th>
<th>Concrete cover $c$</th>
<th>Transverse reinforcement $A_s$</th>
<th>$\Sigma A_s/A_c$ (corresponding to the lap)</th>
<th>$\tau_{\text{max,norm}}$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>BEV1</td>
<td>40</td>
<td>5 · Ø</td>
<td>1.5 · Ø</td>
<td>Ø 14 / 13</td>
<td>1.0</td>
<td>10.1</td>
</tr>
<tr>
<td>BEV8</td>
<td>40</td>
<td>5 · Ø</td>
<td>1.5 · Ø</td>
<td>Ø 8 / 13</td>
<td>0.5</td>
<td>8.8</td>
</tr>
<tr>
<td>BEV14</td>
<td>40</td>
<td>5 · Ø</td>
<td>1.5 · Ø</td>
<td>-</td>
<td>0.0</td>
<td>8.7</td>
</tr>
<tr>
<td>BEV19</td>
<td>40</td>
<td>5 · Ø</td>
<td>1.5 · Ø</td>
<td>Ø 8 / 7</td>
<td>0.5</td>
<td>10.0</td>
</tr>
</tbody>
</table>

4 Anchorage tests

Anchorages of reinforcement with large diameter bars at beam ends are currently investigated by means of six beam tests at RWTH Aachen University. The test parameters comprise the anchorage length, induced cracks, the combination of bar diameters and the confinement both by concrete cover and transverse reinforcement. Table 2 gives an overview of the test parameters. Twelve tests will be conducted with six beams tested at both sides (Figure 4-1).

![Figure 4-1: test setup anchorage test V3-1](image)

The required anchorage length depends on the steel stress at the beam end. In this test series, an investigation of beam ends with and without induced cracks at the front edge of the support is planned. The first tests V3-1 and V3-2 were cast with a vertical metal sheet at the position of the front edge of the support. Both tests had 60 mm concrete cover, 40 mm bar distance (representing the minimum bar distance) and a reinforcement ratio $A_s / A_{c,\text{eff}} = 6\%$.
Table 4-1 Anchorage test parameter

| V1-1  | 30  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 2 · φ |
| V1-2  | 50  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 2 · φ |
| V2-1  | 30  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 1 · φ |
| V2-2  | 50  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 2 · φ |
| V3-1  | 30  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 2 · φ |
| V3-2  | 50  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 2 · φ |
| V4-1  | 30  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 2 · φ |
| V4-2  | 50  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 2 · φ |
| V5-1  | 30  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 2 · φ |
| V5-2  | 50  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 2 · φ |
| V6-1  | 30  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 2 · φ |
| V6-2  | 50  | 40  | lb,rqd,m | x   | 2 Ø 8 mm | Ast = 0 | 2 · φ |

All tests will be conducted with a target concrete strength of 30 N/mm². The mixture is given in Table 4-2. The 40 mm reinforcement bars had a 570 N/mm² yield strength.

Table 4-2: Concrete mixture

<table>
<thead>
<tr>
<th>Cement [kg/m³]</th>
<th>Water [kg/m³]</th>
<th>Plasticizer [kg/m³]</th>
<th>Aggregate (sum) [kg/m³]</th>
<th>Agg. size 0 – 2 [kg/m³]</th>
<th>Agg. size 2 – 8 [kg/m³]</th>
<th>Agg. size 8 – 16 [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>285</td>
<td>180</td>
<td>1.0</td>
<td>1869</td>
<td>748</td>
<td>467</td>
<td>654</td>
</tr>
</tbody>
</table>

During the tests, the deflection, the crack opening at the induced cracks and the slip at each reinforcement bar were measured with displacement transducers. Strain gauges were applied to the reinforcement bars to record the strain along the anchorage length and the stirrup strain where present. The transverse concrete strain is measured with strain gauges and extensometers (Figure 4-2).

Figure 4-2: instrumentation anchorage test V3-1
The mean bond strength was calculated according to Eurocode 2\(^1\) for \(f_{cm} = 30\) N/mm\(^2\).

\[
f_{bm} = 2.25 \cdot f_{ctm} = 2.25 \cdot 0.3 \cdot 30^{2/3} = 6.51\text{ N/mm}^2
\]  

(4-1)

The experimental bond strength of the 40 mm reinforcing bars was obtained from previous beam-end tests as well. The maximum bond strength for a single bar was 10 N/mm\(^2\), while two adjacent bars only reached 6.0 N/mm\(^2\). These bars had 80 mm clear spacing, since the test cylinders were too wide for a 40 mm clear spacing. Beam-end tests with transverse pressure showed considerably higher bond stress for anchored bars. While the bond strength is presumably lower than 6.0 N/mm\(^2\) for the vicinity of the adjacent anchored bars in the anchorage test specimen, the bond strength was presumed to increase for the transverse pressure at the beam end.

The anchorage length was calculated with the additional force from the shear model \(\Delta F_{Ed} = 736\) kN derived from the ultimate cylinder load of 1600 kN resulting in the following value:

\[
l_{b,req} = \Phi \cdot \sigma_s / (4 \cdot f_{bm}) = 40 \text{ mm} \cdot (736\text{ kN} / 37.7\text{ cm}^2) / (4 \cdot 6.51 \text{ N/mm}^2) = 300\text{ mm}
\]  

(4-2)

For the transverse pressure, the anchorage length was reduced by the factor 2/3 resulting in 200 mm. The positive effect of transverse reinforcement positioned in test V3-2 was not taken into account in the anchorage design.

5  Test results

In both tests, the anchorage failed by splitting of the concrete cover at the beam end. The slip of the three reinforcement bars was up to 5 mm at the unloaded end. The shear crack inclination at the front edge of the support was about 35° (Figure 5-1 and Figure 5-2). For the considerable length of the crack inducing steel sheet, shear cracks appeared at the upper beam depth commencing from the steel sheet. Despite the wide shear cracks, the ultimate failure was caused by anchorage failure.

With 6.5 N/mm\(^2\) bond strength, the calculated shear load was 1067 kN. This load was nearly reached in test V3-2 with two 8 mm stirrups in the bond zone (\(V_{ult} = 987\) kN). Test V3-1 without confining reinforcement only reached 64 % or 680 kN respectively.

Figure 5-1 Crack pattern test V3-1  
Figure 5-2 Crack pattern test V3-2
For the evaluation of the bond zone and a comparison with the previously conducted beam-end tests, bond-strength slip curves were derived. While the slip was measured with displacement transducers at each reinforcing bar, the bond strength had to be deduced indirectly. The distribution of bond stress is not linear along the bond length, but for comparability, a mean bond strength will be considered for a first evaluation (equation (5-1)).

\[
\tau = \frac{\phi \cdot \sigma_s}{4 \cdot L_b} = \frac{40 \text{ mm} \cdot \sigma_s}{4 \cdot 200 \text{ mm}}
\]  

The mean bond strength may be derived from the steel stress measured at the induced crack at the front edge of the support, from the crack opening at the same position or from the calculated additional force from the shear model. The bond strength was supposed to be calculated from the steel stress measured with strain gauges at the loaded end of the anchorage, but some strain gauges could not be evaluated for large deformations. The derived steel stress is given by equation (5-2).

\[
\sigma_s(\varepsilon_s) = \varepsilon_s \cdot E_s = \varepsilon_s \cdot 200,000 \text{ N/mm}^2
\]  

Alternatively, the steel stress can be calculated by the crack opening. Neglecting the concrete contribution \( \varepsilon_{cm} \), the steel stress is given by equation 5-3. The crack opening \( w \) is the sum of the bending crack width \( w_{cr} \) and the reinforcing bar slip \( \Delta s \). The crack opening \( w \) was thus reduced by the slip \( \Delta s \) measured at the unloaded end of the anchorage in equation (5-3). When calculating the bond strength by the crack opening, the crack spacing \( s_{cr} \) has to be taken into account. The crack spacing corresponds with the distance between the steel stress minima, which was defined as the sum of the anchorage length (where steel stress is not present at the end) and half the distance to the...
adjacent bending crack (Figure 5-5). The bending crack distance was 300 mm and the anchorage length was 200 mm.

\[
\sigma_s(w) = \frac{\Delta l}{1 \cdot E_s} = \frac{w_{ci} \cdot E_s}{(s_{ci} \cdot E_s)} = \frac{(w - \Delta s)}{(s_{ci} \cdot E_s)} = \frac{(350 \text{ mm} \cdot 200,000 \text{ N} / \text{mm}^2)}{(200 \text{ mm}^2)} \quad (5-3)
\]

The third method for the calculation of the steel stress \(\sigma_s\) is the shear model approach. Equation 5-4 gives the derivation of steel stress at the front edge of the support from the additional force \(\Delta F_{Ed}\).

\[
\sigma_s(\Delta F_{Ed}) = \frac{\Delta F_{Ed}}{\Sigma A_s} = \frac{4 \cdot \Delta F_{Ed}}{3 \cdot \pi \cdot \phi^2} = \frac{4 \cdot V \cdot \cot \theta}{6 \cdot \pi \cdot \phi^2} \quad (5-4)
\]

Figure 5-6 illustrates the calculated steel stress from the measured steel strain, crack opening \(w\) and the calculated additional \(\Delta F_{Ed}\).

![Figure 5-5: Crack-spacing assumption for the anchorage zone](image)

![Figure 5-6: steel stress at the front edge of the support in test V3-2](image)

For the inconsistency of stress results, load-slip curves were compared instead of bond-strength slip curves. The shear load-slip curves for both tests are given in Figure 5-7.

![Figure 5-7: shear load – slip curves in tests V3-1 (left) and V3-2(right)](image)

Despite the splitting of the concrete cover, the anchorages showed high residual load bearing capacities, since transverse pressure and - in test V3-2 - transverse reinforcement was present. The transverse reinforcement stress was well above the yield strength of the 8 mm reinforcing bars in the anchorage.
6 Discussion

While four or five rules for the definition of confining transverse reinforcement in bond zones with large diameter bars appear too numerous and unwieldy, the conducted tests proved the clear necessity of stirrups in anchorages and laps.

Beam tests with anchorages and laps are time consuming and costly, therefore, beam-end tests were previously conducted to investigate whether beam-end test results correspond with beam-test results for different parameters. While the effects of concrete strength and concrete cover in laps and beam-end tests are comparable, the effects of transverse reinforcement on bond strength and bearing capacity of laps and anchorages, respectively, differ considerably.

Whereas Eurocode 2\(^1\) defines an anchorage length commencing from the front edge of the support, the true anchorage length of the longitudinal reinforcement in beams begins at the last bending crack from the support. The reinforcement force to be anchored consists of the bending load and the additional force from the shear model. The bending moment equals zero at the support of the single-span beam and the additional force becomes decisive for the anchorage design. Since the exact position of the last bending crack from the support is randomly distributed, the exact reinforcement stress at the anchorage is not known. For this reason, an induced crack was positioned in the described anchorage tests. The stress at the anchorage-length beginning could not be derived from the deformation measurements clearly, since the crack opening, the reinforcement stress measured with strain gauges and the calculated additional force from the shear model gave differing results.

7 Conclusion

In a current research project, the additional rules for large diameter bars in Eurocode 2\(^1\) are investigated. The additional rules for confining transverse reinforcement in bond zones are numerous and unwieldy. The test programme to analyse the effects of bar diameter, concrete strength, necessary confinement and surface reinforcement for crack control comprises beam-end tests, four-point bending tests with laps and three-point bending tests with anchorages. While the beam-end tests did not show a clear increase in bond strength for confining reinforcement, the laps and anchorages showed a considerable increase in bearing capacity where transverse reinforcement was present. Although the anchorage tests were conducted with the smallest possible transverse reinforcement, the bearing capacity increased by 45 %. The pending anchorage tests will be presented at ConSC 2017.

8 Acknowledgement

The work presented in this paper is part of a collaborative research programme of the Institutes of Structural Concrete of RWTH Aachen University, Technical University Kaiserslautern and Technical University Braunschweig. The research project was funded within the programme for Sponsorship by Industrial Joint Research and Development (IGF, number 18821 N/1) of the German Federal Ministry for Economic Affairs and Energy based on an enactment of the German Parliament.
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EFFECT OF CORROSION OF REINFORCEMENT IN CONCRETE ON PULL-OUT CAPACITY

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ABSTRACT

Chloride-induced corrosion of steel reinforcement in concrete is one of the major causes for deterioration of reinforced concrete (RC) structures. RC structures exposed to aggressive environmental conditions, such as structures close to the sea or highway bridges and garages exposed to de-icing salts, very often exhibit damage due to corrosion. Therefore, to predict durability of RC structures it is important to have a numerical tool, which is able to predict corrosion processes and their consequences for the structural safety. It has been demonstrated that transport of corrosion products in cracked concrete and its effect on corrosion induced damage significantly influences the pull-out capacity of the reinforcement. However, the controlling parameters which describe the transport of corrosion products through cracks has not been quantified by any experimental or theoretical study. In the present paper, better insight into the impressed current technique, frequently used to induce reinforcement corrosion before testing the pull-out capacity, is given. Recently performed experiments under accelerated conditions are simulated and a relationship between diffusivity of corrosion compounds and crack width is proposed, coupling the hysteretic moisture behavior with the transport of corrosion products in cracks.

1 Introduction

Chloride-induced corrosion is considered as one of the major concern for durability of reinforced concrete (RC) structures. Especially vulnerable are structures located in coastal marine environment or highways and garages treated with de-icing salts during winter seasons¹,². Consequences of chloride-induced corrosion of steel reinforcement have negative effects on structural behavior and involve several aspects related to the life cycle of the structure, such as serviceability, safety and structural performance. The main effect of the corrosive attack is manifested as cracking and spalling, due to the expansion of the corrosion products, which have a greater volume than the original steel. As chloride-induced corrosion is one the major cause of deterioration of RC structures, especially bridges, deterioration prediction models of corrosion process in structural elements is essential for an efficient management of the structures. Therefore, challenging task is to develop and improve a numerical tool, which can realistically predict corrosion processes and the related
mechanism of deterioration in RC structures, supporting the service life prediction of damaged and undamaged structures.

Currently, there are a number of models in the literature, which are able to simulate processes before and after depassivation of reinforcement in uncracked concrete\(^1\)\(^,\)\(^3\)\(^,\)\(^4\). For such computational models to be considered as realistic, chemo-hygro-thermo-mechanical processes have to be coupled with mechanical processes and also vice versa. Presently, there are only a limited number of coupled 3D chemo-hygro-thermo-mechanical models capable to realistically predict corrosion process in cracked concrete\(^5\)\(^-\)\(^8\). Furthermore, moisture transport coupled with transport of corrosion products in cracked concrete and its effect on the corrosion-induced damage has not been addressed by any model so far, i.e. the existing models do not account for the transport of water and solution species that are involved in the process of corrosion.

Numerical and experimental results on beam-end test specimen\(^9\)\(^,\)\(^10\) have shown that corrosion induced damage significantly influences the pull-out capacity. In particular, the predicted reduction is higher if the aforementioned transport of corrosion products is accounted for by the numerical model. However, the main problem in defining such processes is quantifying the controlling parameters which lead to realistic results. Therefore, additional theoretical and experimental work is needed on this subject. In the here presented work, the hysteretic moisture behavior is coupled with the transport of corrosion products in cracks and a relationship between diffusivity of corrosion compounds and crack width is proposed. The model and the proposed improvements are validated for one of the geometry examined in the experimental tests, under laboratory controlled conditions.

2 Chemo-hygro-thermo-mechanical-model

A surface layer of ferric oxide covers and protects the steel in concrete. Upon this layer being damaged or depassivated, the corrosion of reinforcement in concrete can be activated. Depassivation of the protective layer can occur by reaching a critical threshold concentration of free chloride ions near the reinforcement bar surface or as a consequence of carbonation of concrete\(^1\). Here is discussed only the chloride-induced type of corrosion that is generally the more severe one. Corrosion of steel in concrete is an electrochemical process dependent on the electrical conductivity of the concrete and steel surfaces, presence and conductivity of electrolyte in the concrete and the concentration of dissolved oxygen in the pore water near the reinforcement\(^3\)\(^,\)\(^11\)\(^,\)\(^12\). The developed 3D chemo-hygro-thermo-mechanical model couples the above mentioned physical and electrochemical processes with the mechanical behaviour of concrete (damage). In the present paper only a short overview of the model is presented. For more informations see Ožbolt et al.\(^5\)\(^,\)\(^6\)\(^,\)\(^13\)\(^,\)\(^14\)\(^,\)\(^15\) and Oršanić\(^10\).

2.1 Non-mechanical processes after depassivation of reinforcement

The corrosion of steel is activated with the depassivation of the steel reinforcement in concrete. The non-mechanical processes important for the propagation stage of steel corrosion in concrete are: mass sinks of oxygen at the steel surface due to cathodic and anodic reactions, the flow of electric current through the conductive pore solution and the cathodic and anodic potentials. The electric
current through the electrolyte is a result of motion of charged particles and, if the electrical neutrality of the system and the uniform ion concentration are assumed, can be written as:

\[ i = -\sigma(S_w, p_{con})\nabla \Phi \]  \hspace{1cm} (1)

where \( \sigma \) is the electrical conductivity of the concrete and \( \Phi \) is electric potential\(^{16}\).

The equation of electrical charge conservation, if the electrical neutrality is accounted for and the electrical conductivity of concrete is assumed as uniformly distributed\(^{16}\), reads:

\[ \nabla^2 \Phi = 0 \]  \hspace{1cm} (2)

The rate of rust production \( J_r \) (kg/m\(^2\)s) and mass of hydrated red rust per related surface area \( (A_r) \) of rebar \( m_r \) (kg), respectively, are calculated as\(^{17}\):

\[ J_r = 5.536 \times 10^{-7} i_a \]

\[ m_r = J_r \Delta t A_r \]  \hspace{1cm} (3)

where \( \Delta t \) is the time interval in which the corrosion is taking place and \( A_r \) is the corresponding surface of the steel reinforcement. The coefficient of proportionality between the anodic current density \( i_a \) and rate of rust production \( J_r \) is calculated using the stoichiometry of chemical reactions and Faraday's law\(^6,13,14,17\).

Recent experimental investigations\(^9,18\) have shown that the penetration of corrosion products into the pores and their relatively large ingress through the radial cracks, generated around the bar, has a significant effect on the development of corrosion-induced damage. The influence can be summarized as: (1) The distribution of rust and radial pressure over the anodic surface is not uniform and (2) Damage due to expansion of products is less pronounced than if they were all deposited locally at the corroding reinforcement surface.

The distribution of corrosion products (red rust) \( R \) (kg/m\(^3\) of pore solution) into the pores and through the cracks in concrete is mathematically formulated as a convective diffusion problem:

\[ \theta_w \frac{\partial R}{\partial t} = \nabla \cdot \left[ \theta_w D_r \nabla R \right] + D_w (\theta_w) \nabla \theta_w \nabla R \]  \hspace{1cm} (4)

in which \( D_r \) is the diffusion coefficient (m\(^2\)/s) of corrosion products. It is important to note that the previous equation does not directly describe the transport of red rust, but rather the distribution of the rust formed in the concrete pores and cracks as a consequence of soluble species (which can dissolve in concrete pore solution and subsequently migrate in the pores and cracks) reacting with oxygen in the pore water\(^{18}\).

### 2.2 Chemo-hygro-thermo-mechanical coupling

The microplane model for concrete with relaxed kinematic constraint\(^{19}\) is applied in the mechanical part of the model. The main advantage of the microplane model is that the material response is calculated based on the monitoring of stresses and strains in different predefined directions.
Integrating uni-axial microplane stresses in a thermodynamically consistent way, from a known macroscopic strain tensor, it is possible to calculate a macroscopic stress tensor. In the finite element analysis, cracks are treated in a smeared way, i.e. a smeared crack approach is employed. To assure the objectivity of the results with respect to the size of the finite elements, crack band theory is used. One-dimensional corrosion contact elements are employed to account for the inelastic strains due to the expansion of corrosion products. They are placed radially around the reinforcement bar surface and their main function is to simulate the contact between reinforcement and the surrounding concrete. The inelastic radial expansion due to corrosion, \( \Delta l_r \), is calculated as:

\[
\Delta l_r = \frac{r}{A_r} \left( \frac{1}{\rho_r} - \frac{r}{\rho_s} \right)
\]

where \( \rho_r \) and \( \rho_s \) are densities of rust and steel, respectively, \( r \) is the ratio between the mass of steel \( (m_s) \) and the corresponding mass of rust \( (m_r) \) over the related surface of reinforcement \( A_r \) that corresponds to the contact element. Note that the ratio between densities of steel and rust depends on the type of corrosion products and can vary from 2 to 7.

3 Reduction of pull-out capacity due to chloride-induced damage

Investigation of generated crack patterns and crack widths due to the corrosion induced damage, as well as their influence on the pull-out capacity of the rebars were experimentally observed by Fischer and numerically validated by Ožbolt et al. and Oršanić.

Figure 1: Predicted and measured results for beam-end specimen with reinforcement bar of 12 mm and cover of 20 mm and reinforcement bar of 16 mm and cover of 35 mm (a,b): relative pull-out capacity as a function of the average corrosion penetration.

Particularly, it is shown that the transport of corrosion products has a considerable influence on the predicted crack pattern leading to reduction of the corrosion induced damage as observed in the
experimental tests. In Fig. 1, the predicted and experimentally measured (average) pull-out capacities and average crack widths on beam-end test specimens with two different cover/reinforcement diameter ratio are shown as a function of the average corrosion penetration\(^9,15\). As can be seen, due to the larger crack widths (damage), the decrease of the pull-out capacity is higher if no transport of corrosion products is accounted for. However, it has to be noted that the diffusivity of the corrosion products is determined by calibrating the model predictions with the experimental results obtained by Fischer\(^9\). Therefore, the diffusivity coefficient is taken in a qualitative sense, bringing to light the need of additional experimental and numerical investigations in order to make a more accurate calibration of the employed parameter (more details can be found in Ožbolt et al.\(^15\) and Oršanić\(^10\).

4 Accelerated corrosion test and numerical analysis

4.1 Geometry, material properties and test setup

Purpose of the experimental and numerical work is the improvement, calibration and validation of the existing CHTM model, with particular reference to the transport of corrosion products into concrete pores and cracks as well as chloride-corrosion induced damage under various environmental conditions and for different geometries. With the help of experimental investigations it is possible to quantify the induced damage (crack width) and the distribution of the corrosion products in concrete for different corrosion stages of reinforcement bar, which can highly influence the pull-out capacity of embedded reinforcement. Other aim of the tests is to have a better insight into the impressed current or potential technique, which is in engineering frequently used to induce reinforcement corrosion. Particular attention is paid to the characterization of the corrosion products formed under different applied potential and exposure conditions. In order to properly evaluate the results in case of anodic polarization and make them comparable with the natural conditions, formed corrosion products and their distribution around the reinforcement in the described experiments were object of examination.

Concrete was manufactured at the Material Testing Institute (MPA) of Stuttgart, using Portland cement CEM I 42.5 N and water/cement ratio of 0.6. Siliceous sand and aggregates were used with a maximum aggregate size of 8 mm. Approximately 2.5 % of NaCl by weight of cement was added to the mixture to accelerate depassivation of steel and improve the electrical conductivity of concrete. The average uniaxial compressive strength (three specimens) is \(f_c = 40\) MPa. Based on \(f_c\) the concrete properties were estimated as: Young’s modulus \(E_c = 31.6\) GPa, Poisson’s ratio \(\nu = 0.18\), tensile strength \(f_t = 3.10\) MPa and fracture energy \(G_F = 55\) J/m\(^2\). Mass density of concrete is assumed to be \(\rho_c = 2400\) kg/m\(^3\). The reinforcement bar is normal steel with the following properties: Young’s modulus \(E_c = 210.0\) GPa, Poisson’s ratio \(\nu = 0.33\) and mass density \(\rho_s = 7800\) kg/m\(^3\). In order to accelerate the corrosion process, an imposed constant potential was applied between the rebar (anode) and a cylindrical shell of platinized titanium mesh (cathode), located around the lateral surface of the specimen. During the experiment readings of current were continuously recorded every 30 minutes measuring the drop of voltage, over a resistance unit of 10 \(\Omega\), and were performed
by a digital multimeter data acquisition and data logging system, KEITHLEY 2701. To electrically connect the external titanium mesh with the concrete surface (see Fig. 2), four stripes of conductive foam 1.5 cm wide, at 90° to each other, were used.

Because of the extensive experimental program and numerous specimens, here are discussed only the results obtained on cylindrical specimens of 50 mm with reinforcement bar of 8 mm under an imposed potential of 100 mV and exposed to wetting/drying conditions. The duration of experiments was approximately 13 months during which the 1st stage was reached after 143 days, the 2nd after 269 days and the 3rd after 390 days.

Figure 2: Geometry of the specimen and details of the accelerated corrosion tests (all in mm).

4.2 Numerical analysis of experimental tests

The main aim of the numerical study is to verify and calibrate the above discussed 3D chemo-hygro-thermo-mechanical model for the case of accelerated corrosion. Furthermore, it should be investigated the influence of water content and concrete conductivity on current density and crack pattern. Moreover, the influence of the imposed electric potential and related current density on the transport of rust and related crack pattern should be studied. In contrast to mechanical part, in the non-mechanical part of the model transient finite element analysis based on the direct implicit integration scheme is carried out. It is assumed that the entire length of the steel bar is activated as anode (depassivated) at the start of the analysis. Consequently, only water content, electric potential, current density at the surface of reinforcement, transport of rust through pores and cracks of concrete are computed. Note that corrosion rate at the surface of reinforcement is calculated from computed electrical current density according to Eq. (1). The parameters of the hysteretic moisture model were calibrated from experimental results obtained for the investigated geometry. The used sorption isotherms (concrete w/c = 0.65) are plotted in Fig. 3a and the corresponding parameters are listed in Tab. 1 (isothermal conditions, temperature = 20°C). In Fig. 3b is also illustrated the change of relative humidity and water content during 7 weekly cycles of wetting and drying as result of the numerical simulation, performed for 100 mV according to the wetting/drying regime exposure of the experimental tests. The water content is expressed in terms of mass water content and the numerical data is taken for the depth of 21 mm from the exposed surface, which corresponds to the interface...
between the steel reinforcement and concrete. Based on the experimental values of current density, the analyses are assumed to start at 60% RH from the adsorption curve. In the first wetting phase there is a large increase of water content, following the adsorption isotherm, before showing hysteretic behavior (scanning loops), which is more pronounced in the presence of concrete damage. The predicted damage influences the non-mechanical processes through the continuous update of the model parameters, such as water vapour permeability, whose dependency on crack width is based on experimental results for water permeability in cracked \cite{21, 22}.

Figure 3: Input main adsorption and desorption curves from the experiments by Hansen \cite{23} for concrete with \(w/c=0.65\) (a) and calculated scanning curves during the 7 weekly cycles of wetting and drying at the depth of 21 mm before and after cracking, under an imposed potential of 100 mV (b).

Furthermore, electrical conductivity of concrete (see Tab. 1) was calibrated based on the experimentally measured average current density and from the experimentally measured conductivity on fully saturated concrete cube specimens using the “Two Electrodes Method” (TEM). The conductivity is relevant for the computation of current density, especially in chloride contaminated concrete \cite{24}. As discussed in Ožbolt et al. \cite{13-15} for the corrosion induced damage the expansion factor of rust and its transport through concrete play an important role. In the used model transport of rust through pores of concrete and cracks is modeled as diffusion-convection governed process (mathematical interpretation). Note that from the physical point of view the rust is not transported, instead, ions are transported and corrosion products are generated in the pores around the reinforcement bar and in cracks.

In order to correctly interpret the results in case of anodic polarization and make them comparable with the natural conditions, corrosion products formed in the presented experiments were examined with Raman spectroscopy. Analysis have shown the presence of a dense layer of products, which is mostly constituted of one or several mixed oxy-hydroxides. The major corrosion products of this layer were identified as goethite (\(\alpha\)-FeOOH) in case of 100 mV (for more details, see Sola et al. \cite{25}). Considering the above mentioned investigations on morphology and distribution of corrosion products in the vicinity of the reinforcement surface, the volume expansion factor of rust is assumed to be congruent with \(\alpha_r = \rho_s/\rho_r = 1.9\) in accelerated conditions (goethite). The relation between diffusivity of rust and crack width is obtain based on the calibration of the experimental data. The
initial diffusion coefficient for un-cracked concrete, obtained by comparing the experimental corrosion time for cracking, is set to $D_r = 21.0 \times 10^{-16} \text{m}^2/\text{s}$.

Based on the results of the performed numerical analyses, discussed in detail in the next sections, it can be concluded that the diffusivity of corrosion products increases with the increase of the crack width, following a logarithmic trend (see Fig. 4).

$$\text{Table 1: Electrical conductivity of concrete (w/c=0.6) \left[ 10^{-3} \Omega^{-1} \text{m}^{-1} \right]}$$

<table>
<thead>
<tr>
<th>Saturatio n</th>
<th>50</th>
<th>55</th>
<th>60</th>
<th>65</th>
<th>70</th>
<th>75</th>
<th>80</th>
<th>85</th>
<th>90</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_{\text{natural}}$</td>
<td>2.75</td>
<td>3.00</td>
<td>4.28</td>
<td>8.70</td>
<td>9.52</td>
<td>10.50</td>
<td>11.50</td>
<td>12.50</td>
<td>13.50</td>
</tr>
<tr>
<td>$\sigma_{\text{accelerated}}$</td>
<td>8.75</td>
<td>9.54</td>
<td>13.61</td>
<td>27.67</td>
<td>30.27</td>
<td>33.39</td>
<td>36.57</td>
<td>39.75</td>
<td>42.93</td>
</tr>
</tbody>
</table>

As can be seen, no change of diffusivity is assumed for water content lower than 60%. At higher level of saturation (between 60% and 70%), a progressive ingress of products is expected. On the other hand, for values of water content between 70 and 100%, the influence of the water content becomes stronger, leading to a large ingress of products in cracks. In this way, the mechanism of transport of corrosion products can be considered as diffusive dominant, hypothesis which make sense considering also the low gradients of relative humidity predicted in the cracks.

Figure 4: Diffusivity of corrosion products as a function of the crack width and level of water saturation.

It has to be noted, that the initial value $D_{r0}$ ($S_w \leq 60\%$) plotted in Fig. 4, has to be considered as a qualitative value, since the initial diffusion in pores and voids in the vicinity of the reinforcement can be significantly influenced from water content and imposed current density.

### 4.3 Comparison between numerical and test results

The average experimental and numerical values of anodic current density-history curves for investigated specimens with imposed potential of 100 mV for the investigated is plotted in Fig. 5. The experimental data are average values on the anodic surface area calculated using measured current intensity. As can be seen, the numerical model is able to realistically simulate anodic current density time history curve.
Figure 5: Comparison between numerical and experimental results in terms of average current density in time, for the investigated specimens exposed to wetting/drying conditions

The analysis and the experiment exhibit sudden increase of current density that is related with sudden wetting of concrete cylinder (see hysteretic behavior of concrete in Fig. 3). Subsequently the water content decreases and current density gradually decreases as well (6 days drying). It can be noted that the moisture content and the anodic current density are strongly influenced by the corrosion induced cracks, which leads to ingress of water into specimen and increase of saturation in un-cracked concrete. Consequently, with increasing wetting-drying cycles, current density tends to increase. Moreover, as already mentioned, once the crack is generated the water content gradually increases in time. This accelerates the transport of rust from the reinforcement surface through the cracks into direction of concrete specimen surface.

Figure 6: Comparison of the crack pattern between experimental (a) and numerical (b) results (wetting/drying conditions) for the investigated specimens

In Fig. 6, the illustration of the predicted corrosion induced damage and the experimentally observed crack pattern is shown. The results indicate the ability of the model to replicate the crack pattern in case of very thin concrete cover, in which the change of environmental conditions plays an important role and has significant influence on the transport of corrosion products.
Additionally, in Fig. 7 the sum of the average calculated widths of the cracks on the external surface of the concrete specimen are plotted as a function of the average corrosion penetration of the reinforcement bar. The computed increase of crack width as a function of the corrosion penetration shows a nice agreement with the experimental obtained data. Furthermore, in Tab. 2 the comparison between the experimental and numerical results is also shown in terms of cracking time $t_{cr}$ and critical corrosion depth $x_{corr}$.

![Graph showing predicted and measured values of crack width as a function of corrosion penetration.](image)

**Figure 7:** Predicted and measured values of the sum of the average crack widths as a function of the average corrosion penetration for the investigated specimens exposed to wetting/drying conditions

**Table 2:** Comparison between numerical and experimental results for the investigated specimens in terms of cracking time $t_{cr}$ and critical corrosion depth $x_{corr}$

<table>
<thead>
<tr>
<th></th>
<th>Experimental test</th>
<th>FE analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_{cr,exp}$</td>
<td>(days)</td>
<td>$t_{cr,num}$</td>
</tr>
<tr>
<td>$x_{corr,exp}$</td>
<td>(µm)</td>
<td>$x_{corr,num}$</td>
</tr>
<tr>
<td>22</td>
<td>9.8</td>
<td>24</td>
</tr>
</tbody>
</table>

As can be seen, after a certain threshold of corrosion penetration, the calculated increase of crack width slows down due to the transport of corrosion products through cracks. This effect is still largely neglected in the current modeling and can significantly influence the reduction of pull-out capacity, therefore it should be included in the evaluation and calibration of analytical and mathematical models.

5 Conclusion

In the present paper the effect of reinforcement corrosion on pull-out capacity is highlighted by means of experimental tests and numerical analysis. In order to bring more light on the influence of the transport of corrosion products through pores and cracks on the predicted pull-out capacity, accelerated corrosion of steel reinforcement embedded in concrete cylinders are performed. Based on the results the following conclusions can be drawn out. (1) The recently proposed coupled 3D chemo-hygro-thermo-mechanical model is able to realistically simulate the reduction of the pull-out...
capacity and correctly replicate experimental tests on accelerated corrosion of steel reinforcement in concrete. (2) Computed current density and related time of cracking and crack patterns are similar to the experimentally observed ones. (3) The transport of rust through cracks in concrete plays important role in corrosion induced damage of concrete. This is especially true if there is relatively high saturation of concrete. (4) In case of accelerated corrosion different types of products are identified with expansion factor lower of that observed in case of natural corrosion. In particular goethite (αFeOOH) was found in case of imposed potentials 100 mV. (5) Considering that the numerical model is able to reproduce the experimental test in terms of critical corrosion penetration, current density time history, crack pattern and development of induced damage, it can be concluded that the proposed relationship for diffusivity as a function of crack width and water content leads to realistic results. (6) The crack pattern and development of the corrosion induced damage in time corresponds nicely with the experimental results for the entire investigated geometric ratios. It is shown that as consequence of the transport of corrosion products, the increase of the crack widths becomes progressively slower significantly affecting the predicted pull-out capacity. (7) Careful interpretation of the impressed current technique is needed, inducing reinforcement corrosion. Investigations on the type and transport of the corrosion products and relative effect on the induced damage should be considered, in comparison with natural conditions, in order to avoid incorrect conclusions in the evaluation of the pull-out capacity.

6 Acknowledgement

The authors are grateful for the financial supports of “Deutsche Forschung Gemeinschaft” (DFG, Grant Nr. 601984) and “Croatian Science Foundation” (HRZZ, Grant Nr. 9068).

References:


A MODEL FOR BOND IN CONCRETE UNDER CORROSION CONDITIONS

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ABSTRACT

A bond-slip numerical model able to simulate the bond mechanism between steel and concrete also in presence of corrosion is shown. It takes into account different factors as the level of corrosion, bar diameter, concrete strength and presence of lateral confinement. The proposed model is based on the thick-walled cylinder theory so that bond performance is a direct consequence of radial pressure acting on the rebar and coming from the surrounding elements. The elastic contribution of concrete is taken into account as well as the inelastic one. The model is used to simulate pull-out tests and it is also modified to reproduce anchorages like for end-beam tests. The results show the effectiveness of the model to be employed for the performance evaluation of the residual capacity of existing structures.

1. Introduction

Corrosion of reinforcement in concrete is a very widespread issue for reinforced concrete (r.c.) structures; it includes cover cracking, spalling, cross section loss and the modification of the bond mechanism. The latter, that is the force at the interface between steel and concrete, is of outmost importance because it is accountable for the structural performances. Modifications of tension stiffening, anchorage length, ductility and bearing capacity of r.c. structures can be addressed to bond variations. Mechanical models have been developed for bond-slip relationship reproducing bond test with long (beam type) and short embedment length (pull-out). In order to study a local relationship a set of experiments with short embedment length can be used¹ as a set up for the model. There are plenty of references in the scientific literature able to reproduce and predict the bond behaviour in pull-out tests. In Fantilli & Vallini² a one dimensional analytical approach for the simulation of pull-out test mechanism of uncorroded smooth bars is used. Lundgren et al.³ used a similar approach to simulate the bond-slip behaviour of corroded ribbed reinforcement. The Authors introduced an updated version of the CEB-FIP Model Code 1990⁴ characterized by a plastic slip in the first branch of bond-slip relation. Hence, this model is capable to take into account the effect of both corrosion and transversal confinement provided by the presence of stirrups. The effect of corrosion is simulated by means of a progressive reduction of bond-slip relationship from confined to unconfined configuration. Other different models based on finite element procedures, as in Ozbolt et al.⁵, have been also proposed; it is a coupled chemo-hygro-thermo-mechanical model which takes into account...
the reinforcement corrosion through micro electro chemical dynamics. Electrical conductivity of concrete and steel surfaces, concrete embedded electrolyte conductivity as well as dissolved oxygen availability in the pore water near the steel reinforcement are evaluated and integrated in order to come up with the mechanical behaviour of damaged concrete. Hussein & Amleh\textsuperscript{6} presented a bond strength model based on the thick-walled cylinder theory able to consider the presence of reinforcement corrosion. The maximum bond strength derives from a cohesive friction principle in which three contributions are combined: the bond strength contribution due to adhesion between corroded steel bars and the surrounding cracked concrete; the contribution of maximum confining pressure at anchorage bond failure and, eventually, the corrosion-induced internal pressure. The model is unable to simulate the presence of active or passive confinement that undoubtedly leads to different performance.

2 Proposed Model

In this work, a relatively simple numerical model is proposed in order to reproduce the performance of bond in presence of corrosion with pull-out tests. In this model, the assumptions reported in Noghabi\textsuperscript{7}, which allows to evaluate the bar-to-concrete pressure due to bond action, are implemented. Furthermore, the structural effect of corrosion is taken into account. Pull-out and splitting mode failures are simulated and a comparison with experimental tests is shown. The bond mechanism is analytically reproduced in the longitudinal and in transversal direction; the former includes the solution of the equations that governs the relative slip mechanism, the latter is responsible for the concrete contribute to the embedded bar bond capacity.

2.1 Solution for the BVP

In pull-out tests, the mean bond stress along the embedment length and the slip at the unloaded end of the specimen are registered. According to the RILEM (1994) guidelines, the anchored length is short (five times the bar diameters) in order to avoid yielding of the reinforcement before the complete sample failure due to bond. In the anchorage zone, the mean bond stress ($\tau_m$) can be calculated as follows:

\[
\tau_m = \frac{F}{\pi \cdot \phi \cdot l_b}
\]  

(1)

where $F$ is the acting force, $\phi$ is the bar diameter and $l_b$ is the bonded bar length. The mean bond slip relationship is derivable from equation (1) for a range of values of $F$ and corresponding slip $s$. Three analytical equations govern the bond mechanism:

- equilibrium equation of a generic cross section:

\[
F = A_c \varepsilon_c E_c + A_s \varepsilon_s E_s
\]  

(2)

where $A_c$ is the concrete section, $\varepsilon_c$ is the concrete strain, $E_c$ and $E_s$ are the elastic module of concrete and steel respectively, $A_s$ is the steel section and $\varepsilon_s$ is the steel strain;

- equilibrium equation of a generic portion of bare bar:
\[(\varepsilon_s + d\varepsilon_s)E_sA_s - \varepsilon_sE_sA_s = \tau_s\pi\partial dx\]  
(3)
taking into account the formulation for a generic section area of a circle, it follows:
\[
\frac{d\varepsilon_s}{dx} = \frac{4\tau_s}{E_s\partial}
\]
(4)

- compatibility equation at the interface layer:
\[
\varepsilon_s - \varepsilon_c = \frac{ds}{dx}
\]
where \(s\) is the bar slip between steel and concrete.

To solve this boundary problem, it is necessary to introduce a boundary condition that is:
\[
\varepsilon_{s,x=0} = \frac{F}{A_sE_s}
\]
(6)

The boundary value problem is solved by means of Euler’s method for ordinary differential equations. The outcomes for every applied slip at the loaded end are the value of pulling force \(F\) and the functions \(\varepsilon_s(x), \varepsilon_c(x), s(x)\) and \(\tau(x)\). In this way, it is possible to simulate a displacement controlled test. In order to take into account the effect of the lateral pressure due to bar-concrete interaction caused by pull-out induced stresses and corrosion, an extension of this model is proposed.

### 2.2 Analytical model for thick-walled concrete ring

A relevant aspect in pull-out test is the effect of radial pressure that arises between concrete and reinforcement when the mechanical action of bond is present; this is related to the interaction between the ribs and the surrounding concrete. This mechanism produces compressive radial stresses and consequently hoop tensile circumferential stresses that can bring to the cracking of concrete (splitting cracks). In order to take into account this effect an elastic cracked cohesive model is employed: both the strength of uncracked concrete and the cohesive contribution of cracked part are considered. Therefore, the concrete section is divided into two layers: elastic uncracked ring (external) and cracked cohesive part (internal). The cohesive effect of cracked concrete is simulated by means of a double linear softening law proposed by the MC2010:

\[
\sigma_{ct} = \begin{cases} 
  f_{ct} \left(1 - 0.8 \frac{w}{w_1}\right) & \text{for } w \leq w_1 \\
  f_{ct} \left(0.25 - 0.05 \frac{w}{w_1}\right) & \text{for } w_1 < w \leq w_c
\end{cases}
\]
(8)

where \(w\) is the crack width, \(f_{ct}\) is the concrete tensile strength, \(w_1\) and \(w_c\) are two parameters related to concrete fracture energy.

Starting from an equilibrium between internal and external forces, the following equation can be written:
\[
2p_i r_i = 2p_{cr}r_{cr} + 2 \int_{r_i}^{r_{cr}} \sigma_{ct}(r)dr
\]
(9)
where \( r_i \) is the internal radius (nominal radius of the reinforcing bar), \( r_{cr} \) is the radius of the cracked ring and \( \sigma_t \) the residual tensile stress in the cracked concrete according to equation 7 or equation 8. In correspondence of the cracked ring \((r=r_{cr})\) the hoop stresses are equal to concrete tensile strength \((f_{ct})\). The cracking mechanism determines the unloading of the bulk and defines the crack width. Therefore, the displacement is calculated as:

\[
 u_t(r) = 2(\varepsilon_e + \varepsilon_{cr})\pi r = \left(\frac{2\sigma_{ct}}{E_c} + \frac{nw}{\pi r}\right)\pi r
\]  

(10)

where \( n \) indicates the number of cracks. In the cracked ring the tangential displacement is assumed constant and equal to:

\[
 u_t(r_{cr}) = 2\pi \varepsilon_{ct} r_{cr}
\]  

(11)

therefore, in the cracked ring, rewriting eq. 10 with respect to the crack width:

\[
 nw = 2\pi \varepsilon_{ct} \left( r_{cr} - r \frac{\sigma_{ct}}{f_{ct}} \right)
\]  

(12)

It is possible to substitute equations 7 or 8 into equation 12 in order to obtain the expression of the concrete tensile stress in the ring, that is respectively:

\[
\sigma_{ct}(r) = f_{ct} \frac{1 - r_{cr} 2\pi \varepsilon_{ct} / (nw_c)}{1 - r 2\pi \varepsilon_{ct} / (nw_c)}
\]  

(13)

\[
\sigma_{ct}(r) = f_{ct} \frac{nw_1 - 1.6\pi \varepsilon_{cr} r_{cr}}{nw_1 - 1.6\pi \varepsilon_{cr} r}
\]  

(14a)

\[
\sigma_{ct}(r) = f_{ct} \frac{0.25nw_1 - 0.1\pi \varepsilon_{cr} r_{cr}}{nw_1 - 0.1\pi \varepsilon_{cr} r}
\]  

(14b)

where \( w_c \) and \( w_f \) are the concrete parameters related to the material energy fracture property.

In this way, the concrete tensile stress is made explicit and it can be substituted into eq. 9 and afterwards integrated. Nevertheless, once the substitution of equations 14a and 14b is achieved, the following stability requirements should be observed:

\[
r_{cr} < \frac{nw_1}{2\pi \varepsilon_{ct}} \quad \text{and} \quad r_{cr} < \frac{2.5 \cdot nw_1}{\pi \varepsilon_{ct}}
\]  

(15)

### 2.3 The contribution of transversal reinforcement

In order to evaluate the effect of transversal reinforcement an additional contribution should be added to the model according to the following assumptions:

- the transversal reinforcement is transformed into an equivalent cylinder with an equivalent thickness \((t_{s,eq})\);
- stresses in the cylinder are assumed to be constant;
- the steel is assumed to be a linear elastic material;
• square shaped transversal reinforcement is transformed into circular by means of an equivalent radius procedure.

Depending on the position of the crack front and the diameter of the steel cylinder, two situations are possible: the crack front is internal to the cylinder or it is external. In the first case, elastic thick-walled ring theory is applicable and so the tangential deformation at a generic radius \( r \) can be evaluated as follows:

\[
\varepsilon_t(r) = \frac{2p_{cr}r^2}{E_c(r_0^2 - r_{cr}^2)r^2} = \varepsilon_t(r_{cr}) \frac{r_{cr}^2}{r^2}
\]  

(16)

Knowing that \( \varepsilon_t(r_{cr}) = \frac{f_{ct}}{E_c} \), at the transversal reinforcement level \( (r_s) \) the deformation of the steel is known.

When the equivalent steel cylinder is inside the cracked ring, the same assumption on the cracked ring done in the previous section (eq. 11) is applied, thereby:

\[
\varepsilon_t(r_s) = \frac{f_{ct}r_{cr}}{E_cr_s}
\]  

(17)

With the assumption about the steel as a linear elastic material, the equilibrium equation (9) becomes:

\[
2p_i r_i = 2p_{cr}r_{cr} + 2 \int_{r_i}^{r_{cr}} \sigma_{ct}(r)dr + 2E_s \varepsilon_t(r_s)t_{s,eq}
\]  

(19)

### 2.4 Relationship between radial displacement and internal pressure

Pull-out mechanism induces radial displacement because of the presence of concrete struts that arise due to ribs of bars. Furthermore, the radial displacement can be due to the presence of oxides products or to the pull-out mechanism. Oxides products induce internal pressure in consequence of their volumetric expansion. By definition, the tangential displacement is defined as follows:

\[
\varepsilon_t(r) = \frac{v}{r}
\]  

(20)

where \( v \) is the radial displacement. Furthermore, assuming that:

\[
\varepsilon_t(r_i)r_i = \varepsilon_t(r_{cr})r_{cr}
\]  

(21)

and with the definition of the tangential strain corresponding to the cracked radius, the latter can be evaluated as follows:

\[
r_{cr} = \varepsilon_t(r_i)r_i \frac{E_c}{f_{ct}} = v \frac{E_c}{f_{ct}}
\]  

(23)

Therefore, imposing a radial displacement \( v \), from eq. 23 it is possible to evaluate the cracked radius \( (r_{cr}) \) and then the internal pressure (19). Hence, the radial displacement can be induced by the wedging action of the ribs towards the surrounding concrete during a pull-out test. To simulate the radial displacement during pull-out test, two formulations are adopted as much as the types of
failures that a specimen can show during the test. In case of pull-out failure, the following nonlinear relationship is proposed:

\[
v = \begin{cases} 
  v_{s1} \left( \frac{s}{s_1} \right)^{0.4} & \text{for } s \leq s_1 \\
  v_{s1} + \tan \beta \cdot (s - s_1) & \text{for } s_1 < s \leq 2 \text{ mm} \\
  v_{s1} + \tan \beta \cdot (2 - s_1) & \text{for } s > 2 \text{ mm}
\end{cases}
\] (24)

where \(v_{s1}\) is a parameter function of the diameter of the reinforcing steel bar and \(\tan \beta\) is an experimental parameter set equal to \(1.5 \cdot 10^{-4}\). The curve represented in equation 24 consider a substantial constant level of radial displacement between \(s_1\) and 2 mm as the effect wedging effect of the single rib. Hence a constant level of radial pressure due to the test during this step is present. After 2 mm of slip, it is assumed that no further radial displacement is possible.

In case of splitting failure, the radial displacement follows the proposed relationship:

\[
v = s \cdot \tan \gamma
\] (25)

where \(\tan \gamma=0.1\) simulate a sliding plane that goes from top of the rib to the base of the following rib\(^9\).

As previously mentioned, the radial displacement can also be due to the presence of expansive oxides products which originate from the corroded steel bar and push the surrounding concrete outwards. Corrosion of bars can be expressed as a mean value in terms of attack penetration \((x_{ave})\). The attack penetration can be evaluated from the mass loss, assuming a homogenous cross section reduction. The phenomenon is complex because corrosion products behave like a gel that moves through concrete pores, fills cracks when concrete is cracked and finally flows out when it reaches the external surface. Starting from a corrosion penetration \(x_{ave}\) it is possible to evaluate the volume of the oxides product:

\[
\Delta V_r = \alpha \pi [r_b^2 - (r_b - x_{ave})^2]
\] (26)

where \(\alpha\) is the volumetric expansion coefficient of the rust and \(r_b\) is the bar radius. It is assumed that the oxides fill an equivalent thickness \((h_p)\) of concrete before triggering a pressure value in order to take into account the porosity of the concrete matrix. This thickness is assumed to be constant through the cracks that have a triangular shape (in case the cracks remain into the concrete section) or a trapezoidal shape (in case of the cracks exceed the external surface). Thereby, a relationship between the radial displacement and the reduced radius is established as follows:

\[
r_{rb} = \left\{ \frac{1}{1 - \alpha} \left[ (v + r_b + h_p)^2 - \alpha r_b^2 + \frac{(\pi v + 2nh_p)(r_{cb} - v - r_b)}{\pi} \right] \right\}^{1/2}
\] (27)

where \(n\) is the number of cracks and \(v\) is the radial displacement.

### 2.5 Maximum bond strength allowable criterion

A possible criterion able to define the bond-slip local relationship between steel and concrete considers the actual lateral pressure (due to mechanical and chemical effect) to establish the maximum bond strength to be used. A friction-type mechanism is proposed:
\[ \tau_{\text{max}} = c + \mu \sigma_r \]  

where \( c \) is the cohesion and \( \sigma_r \) is the lateral pressure. In absence of experimental data, \( c \) it can be assumed equal to the maximum bond strength of smooth reinforcing bars. The coefficient \( \mu \) represents the friction between steel and concrete, it is assumed to be 0.9 for uncorroded reinforcement. In presence of corrosion, the value decreases starting from the attack penetration that corresponds to the cover cracking \( (x_{cr}) \), according to:

\[ \mu = 0.9 - (x_{ave} - x_{cr}) \geq 0.5 \quad \text{for} \quad x_{ave} \geq x_{cr} \]  

(29)

the friction angle reduction approach has been adopted elsewhere\(^\text{10}\).

Figure 1 Geographical representation of the resisting criterion based on cohesion and friction

Even though the MC2010\(^8\) indicates \( \tau_{\text{max}} \) as a function of the compressive strength of concrete, in the present work the bond strength is a function of the lateral pressure. Hence, in each point of the mesh, the bond-slip curve is an homothetic variation of the local bond-slip relationship. Furthermore, the dominion could be characterized by different confining states along the reinforcing bar axis allowing to consider special conditions such as external sources of confinement.

### 3 Results of the numerical modelling

The proposed model, introduced in the previous section, is applied and calibrated with the output of a pull-out campaign. The specimen type adopted in the present work is the standard pull-out RILEM test. Steel reinforcement is grade B450C according to the Italian Standard. The longitudinal reinforcements are straight bars with 12 mm and 16 mm diameter. Therefore, the embedment length is set equal to 5 times the reinforcement diameter resulting in 60 mm and 80 mm of bonded length respectively; the unbonded zone is guaranteed through a plastic sleeve. The specimens provided with transversal reinforcement have 2 closed stirrups with 8 mm diameter and spacing of 23 mm for specimens with 12 mm diameter as longitudinal reinforcement, whereas the spacing is 50 mm for samples with 16 mm diameter as main bar. All corroded specimens are subjected to accelerated electrochemical corrosion procedure through the application of 200 \( \mu \text{A/cm}^2 \) current density in line with other experiences performed in the past by the same Authors. Furthermore, wet and dry daily procedure is employed to ensure oxygen and water cyclic presence and effectiveness for producing adequately hydrated corrosion products.

The mechanical interaction between reinforcing bar and concrete is related to the lateral displacement \( v_s \) that determines the lateral pressure. Because of the importance of \( v_s \) factor, a sensitivity evaluation is exerted using a \( \pm 20\% \) of its value. On the base of the experimental data related to the crack onset, the volumetric oxide ratio \( (\alpha) \) and the equivalent porosity thickness \( (h_p) \)
were assumed equal to 2 and 20 µm respectively. In the diagrams reported in Figure 2, the model simulations are reported with the discontinuous lines, whereas the experimental curves are the continuous ones. The numerical curves correspond to the different values for \( v_{s1} \) parameter that comes up with the lower bound curve (model_{LB}) and the upper bound curve (model_{UB}). As far as the corrosion takes place, the numerical model_{LB} and model_{UB} reduce their differences; corrosion degradation leads the specimen towards splitting failure and then the rib height become non-influential since that corrosion is the governing phenomenon.

In Figure 2 the comparison between numerical and experimental curves can be observed. Unconfined samples behaviour is well reproduced: corrosion deterioration leads to a sharp reduction of bond performances. The confined samples, instead, show a good bond performance even in presence of severe corrosion demonstrating the effectiveness of the transversal reinforcement. Some numerical simulations overestimate the experimental outcomes, nevertheless in some cases (S16-C00-1/2) the experimental results seem to be under the expected performance because the unconfined pairs (N16-C00-1/2) shown a higher bond.

In Figure 3 the experimental and numerical bond strength for 12 mm embedded bar without confinement and in presence of stirrups, are reported. The specimens are named with their actual corrosion attack penetration expressed in µm rather than their label name. The bond behaviour results to be clearly different in presence of confinement: for the latter, high performances are observable, indeed, even in presence of severe corrosion. On the other hand, unconfined samples show poor bond performances in presence of high levels of corrosion (≥10%); in this case, concrete cover results cracked and, therefore, a small confinement contribution of concrete is present.

Figure 2 Model simulation (discontinuous lines) compared to experimental outcomes (continuous line) for confined and unconfined RILEM specimens
The specimens with corroded reinforcement up to 2% of mass loss, instead, show an increase in performance due to the beneficial effect of corrosion pressure that, in these cases, do not yet crack the concrete cover thickness. The model simulations agree with the behaviour of the specimens, the bond strength results adequate in the most cases. In the case of confined specimens with 10% of nominal level of corrosion, the bond performance is slightly overestimated. Nevertheless, the experimental outcomes should be evaluated carefully since that, in some cases, unexpected low bond performance are observed due to, probably, poor concrete compaction during the casting phase (N12-C00-1, N12-C10-2, S12-C02-1 and S12-C10-1). The aforementioned issue concerns, in particular, the specimens provided with transversal reinforcement which physically obstruct the concrete compaction during the casting.

Afterwards the model is modified to reproduce anchorages like end-beam tests. In the present work, the tests on beam end with inclined shear cracking shape, reported in Coronelli et al.11, are considered. The specimens are reinforced with three ribbed 20 mm diameter steel bars; specimens with and without transversal confinement are employed. The transversal reinforcement for the former group are ribbed 8 mm diameter bars with spacing of 40 mm. The cover-to-bar diameter ratio is 1.5; the bonded length is equal to 210 mm. All of the specimens are subjected to accelerated corrosion, with an average current density of 100 μA/cm² for a period of time necessary to gather a rebar weight loss of 20% (main bar).
The predicted bond strengths fit the experimental outcomes in a reasonable manner. Furthermore, if
the experimental outcomes between uncorroded unconfined and uncorroded confined specimens are
compared, a bond gap is evidenced. The difference can be addressed to the presence of the stirrups,
the proposed model is able to deal with such different structural configuration giving different bond
responses. As far as the corrosion increases, unconfined specimens show a bond strength decay
which is nearly absent in the confined specimens instead. The differences in terms of bond strength
performance highlighted for the uncorroded confined samples between the model and the test, reveal
an increase of scatter of the differences and in particular the model overestimates for low values of
corrosion and underestimates for high levels.

4 Discussion

The pull-out test evidenced the efficiency of the confinement which allows to maintain good bond
performances even in presence of severe corrosion; the confined specimens (series S), indeed, shown
a steady performance with very small bond reduction. The unconfined specimens with smaller
reinforcement (12 mm) take advantage of corrosion up to around 2% showing a bond performance
improvement. After that, the bond efficiency decreases steadily because of brittle failure during the
test (splitting); these specimens were characterized by the presence of corrosion cracks since the end
of the corrosion phase.

A good agreement between the experimental evidences and the model simulations is highlighted,
considering the high scatter and variability of corrosion phenomenon and that also reflects in the
bond tests results. In light of this, and considering all the simplifying assumptions proposed, the
forecast bond performances can be considered satisfactory. The overestimation of the model can be
also addressed to the concrete creep effect that is not implemented and is due to the presence of
imposed deformations due to the oxide expansion. This determines a pressure reduction of the oxides
as well as to the stress relaxation of concrete in time.

The modification of the model allows to simulate different types of tests such as in Coronelli et al.11.
In this case, the bond strength prediction is acceptable both for unconfined and confined specimens.
As far as the latter group is concerned, the bond strengths are different between the samples with
corroded and uncorroded stirrups. The different test performance could be addressed to the massive
presence of oxides because of the corroded transversal confinement that increases the pressure
around the longitudinal reinforcing bars. This particular circumstance is implemented in the model
and, therefore, the bond strengths result to be different (comparison of the column height of the
histograms in figure 4b and 4c), greater in the case of corroded stirrups, in particular.

5 Conclusions

In the present work, the Authors presented a bond-slip numerical model able to simulate the bond
mechanism between steel and concrete also in presence of corrosion. The modelling of the structural
element is performed both in longitudinal and transversal direction so as to establish a three-
dimensional behaviour. In the longitudinal direction, the equations that govern the bond-slip
relationship are solved through Euler’s method which can be easily managed with a numerical
computer program. Transversal modelling takes advantages from the thick-walled-cylinder theory
which allows to consider the mechanical effect of bond mechanism as well as the consequences of
the reinforcing bar corrosion. Possible lateral confinement is implemented in the model smearing the transversal reinforcement into an equivalent steel cylinder. The transversal modelling comes up with a radial pressure that operates on the reinforcing bar surface modifying its mechanical behaviour. Such pressure, indeed, contributes to define the maximum bond strength that could be achieved in the considered section. Therefore, a homothetic variation of a modified bond-slip curve is established depending on boundary condition such as the pressure due to the mechanical action and corrosion, active or passive confinement and the confinement provided by the concrete surrounding the reinforcing bar. Authors’ experimental campaign on pull-out RILEM samples is used to set up the model; the results are satisfactory. The reliability of the proposed model is verified reproducing another experimental test available in literature; it is characterized by a longer embedment length of the reinforcing bar and the different confinement provided. The numerical outcomes simulate the experimental behaviour even though some differences in the bond strength is present in some cases. Nevertheless, taking into account the experimental complexity, variables and the uncertainties of the corrosion phenomena that is coupled with the mechanical variability of the bond performance, the numerical model can be considered as valuable tool for simulation.

6 Acknowledgement

This work was founded with the Italian Ministerial fund PRIN 2015.

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INFLUENCE OF CORROSION OF BRASS METALIZED PLAIN STEEL BARS ON BOND STRENGTH WITH CONCRETE

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ABSTRACT

This paper evaluates the influence of thickness of metallized brass coating on bond strength of smooth bars in concrete. In the experiment, corrosion stability of the coating was evaluated considering the initial corrosion damage caused by alkali pore solution of fresh concrete. Bond strength was evaluated for both normal strength concrete (NSC) and ultra-high performance concrete (UHPC). Further, the corrosion attack on coating was studied microscopically and the relationship between its character and final bond strength was described. The results suggest that corrosion resistance of such coated reinforcement and its bond strength are related to the initial alkalinity of both fresh concretes and the additional alkalinity of UHPC. The results are discussed with regard to the trend of using dispersed steel concrete reinforcement coated with brass in high-performance composites.

1 Problematics of corrosion reinforcement and its coating

Corrosion of concrete reinforcement is, up to this date, still one of the fundamental corrosion problems limiting lifetime of reinforced concrete constructions. Typical reinforcement consisting of carbon steel corrodes in both fresh and curing concrete in passive state with acceptable corrosion rates. This is mostly caused by surplus of Ca(OH)₂ which forms by hydration reaction of silicates, keeping pH of the pore solution above 12.6. To smaller extent, this is also affected by small amounts of K₂O and Na₂O present in cements. High pH value ensures that carbon steel is in the passive state¹,²,³.

Acceleration of corrosion of reinforcement can be caused by two mechanisms and their combination. Transient of steel from passive to active state, resulting in higher corrosion rates, is caused by carbonation. Carbonation is gradual reduction of alkalinity of pore solution by reaction of Ca(OH)₂ with gaseous CO₂ (or other acidic gases such as SO₂ and NOₓ), forming CaCO₃ (resp. other salts). Increase of corrosion rate via passive-active transient can be also caused by contamination of concrete by chloride anions (e.g. from de-icing salts, marine splash zone). Stress effects of voluminous corrosion products cause disintegration the covering layer of concrete thus facilitating further damage to the reinforcement.
There are many possible solutions to this corrosion problem, differing in effectiveness and cost. Some methods lack effectivity (corrosion inhibitors), others cannot be used universally (cathodic protection) and some are unacceptable for financial reasons (reinforcement made of stainless steel, carbon fibres or other non-metallic materials).

From this point of view, conventional carbon steel with coating has a good potential for application. Coatings are universal form of protection with reasonable costs and proven effectivity, however there are still limits. All coatings only prolong time to reach passive-active transient or until the localized attack of steel caused by chlorides takes place. From the economical standpoint, two variants of coatings were tested – organic coatings (primarily epoxy-based) and hot-dip zinc galvanized coatings. Defect-less organic coatings do not react with cement and prolong the time until the underlying steel is attacked. It has been proved that use of this kind of coating on a reinforcement results in reduction of bond strength with concrete of common mechanical properties. Use of zinc-galvanized reinforcement in concrete is sporadic. In strongly alkaline environment (pH ~ 13.0) the zinc corrodes in active states, forming hydrogen can reduce bond strength between concrete and the reinforcement. Forming crystalline corrosion products of zinc (Ca\([\text{Zn(OH)}_3]\)_2·2H_2O) can also have negative effect on bond strength.

In this paper, the authors focused on bond strength of brass-coated reinforcement deposited by conventional metallization technology. The bond strength with concrete as well as coating degradation were studied. Aim of the work was to assess effectivity of this coating with focus on bond strength in NSC and UHPC. Research in this field is motivated by today frequent use of brass-galvanized steel wires as dispersed reinforcement in cement-composite materials to enhance their corrosion resistance.

## Experiment

Experiment was split in two parts. In first part, bond strength was evaluated via the pull-out test. Four sets of samples (4 samples in each set) were tested: steel bar without coating (Fig. 1 – bottom), and bar with brass coating (Fig. 1 – top). Both bar types were cast in NSC and UHPC. By the means of brass metallization, smooth steel bars (type 10 216) were prepared for conventional bond strength tests. Bars were sanded to ensure comparable roughness levels. Composition of brass wire used for coating was measured with XRF (no-calibration method, Axios-PANalytical with Omnian software). Relative-weight composition of the wire was 77 % Cu, 22 % Zn and 0.4 % Fe.

Figure 1: Detail of steel bar surface with brass coating (top) and without any coating (bottom)
Distribution of phases within the coating was evaluated by the means of metallography (manual grinding on abrasive papers P60-P1500, polishing with diamond paste). Fig. 2 phase distribution in metallized brass coating. The coating consists of α-brass, i.e. solution of zinc in copper. Coating was studied in more detail using electron microscope (Tescan LYRA3) equipped with EDS detector. Appearance of undamaged coating on steel bar, further used as a reference, is shown in Fig. 2.

Figure 2: Microstructure of brass coating on wire – light microscopy

Figure 3: Reference appearance of undamaged brass coating on steel bar

3 Results

Bond strength tests were conducted on all sets of samples with identical boundary conditions. Reinforcement bars of 12 mm diameter were pulled out from cubes with edge length of 150 mm. Plot in Fig. 4 shows shear stress and displacement of upper unloaded end of the bar. Plot in Fig. 5 shows relative representation of maximum average bond stresses for individual sets (100 % is always the uncoated set).

Figure 4: Average shear stress dependence on displacement of the unloaded end of the reinforcement

Figure 5: Comparison of maximum stresses
The brass coating was studied in detail prior to testing with focus on thickness and morphology (Fig. 3, 6). The results show usual splat character of coating (this is common for all metallized coatings) with average thickness of 300 μm ± 30 μm. Bond strength measured using the pull-out test showed significant differences in shear stress values: 9.1 for NSC and 17.1 MPa and UHPC for bars without any coating, 3.5 and 14.1 MPa for brass-coated bars (Fig. 4, 5). Reduced bond strength of smooth bar with brass coating in the NSC coating (approx 38 % of maximum value of uncoated reinforcement) can be, similarly to zinc-galvanized reinforcement, explained by coating corrosion with simultaneous hydrogen evolution in fresh and curing concrete. Alkaline electrolyte penetrates easily into the caverns between splats and corrosion attack also takes places outside the external surface. In consequence, the coating can disintegrate after loading (bond-strength test). Damage to the coating caused by corrosion in concrete (in case of the NSC) is shown in Fig. 7. EDS analysis showed calcium in form of precipitated corrosion products – most likely zincates or precipitated surplus of portlandite Ca(OH)₂, partially integrated in the coating. This finding further supports findings about corrosion attack of internal parts of the coating. After splitting the concrete cube, the reinforcement-concrete interface was studied and it was verified that brass coating corrodes with hydrogen evolution which increases porosity of cement (Fig. 8). This fact also contributes to reduction of bond strength.

On the contrary, in the case of similarly coated reinforcement cast in UHPC, the bond strength was not decreased. For the UHPC, the bond strength reduction due to the coating was about 78 %. Detail in Fig. 9 clearly shows that coating was not significantly attacked. The image also shows that disintegration took place above the boundary of the coating (local cohesion fracture). Coating is not significantly attacked and probably there was no hydrogen evolution, like in case of NSC.
Conclusion

The paper proves that corrosion of brass coating can have negative effect on bond strength between reinforcement and normal-strength concrete. Reduction of bond strength is caused by corrosion of coating in fresh and curing concrete resulting in hydrogen evolution and formation of crystalline corrosion products. Layer of the corrosion products have very negative dividing effect, resulting in disbonding of bar in the NSC. Based on the data obtained, the decrease was to as low as 38 % of the initial value.

In case of samples cast in the UHPC, this reduction of bond strength was not confirmed. Values of average shear stress of coated reinforcement were about 78 % of the uncoated samples. Study of the corrosion of the brass coating after bond-strength test showed significant differences compared to the NSC. In the initial stages of UHPC curing, the pore solution is less alkaline and thus not causing significant corrosion damage. The bond strength is preserved due to lack of corrosion products. Motivation of this research was verification of harmlessness of brass coating in dispersed reinforcement in UHPC concretes. This theory was confirmed using both mechanical and microscope analyses.

Nevertheless it must be noted that UHPC hydrates more easily compared to the NSC (UHPC contains higher amount of conventional silicate cement) and it is therefore necessary to study the progress of corrosion processes for longer periods of time. It is also necessary to study the effect of humidity (concrete storage) on corrosion behaviour and bond strength. To achieve more significant results, it is necessary to expand on current data and use statistical analysis.

Acknowledgement

This paper describes results of the GAČR P105/12/G059 project. The test were carried out in laboratories of Klokner Institute, CTU in Prague.
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BOND BETWEEN STEEL AND CONCRETE UNDER FIRE - FROM LABORATORY TESTS TO FIRE PERFORMANCE

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ABSTRACT

The bond between steel rebar and concrete plays an important role in the behavior of the RC structures, since it assures the stress transfer between the two materials. Its function is generally compromised during fire and needs to be assessed after the fire accident. The tests to evaluate the degradation of bond due to high temperatures are mainly performed using relatively simple pull-out specimens exposed to slow heating until reaching a steady state. The tests are often limited to relatively small bonded length. Since such tests do not necessarily reflect the actual conditions found in RC beams, there is a need to relate these results to the realistic bond behavior of reinforcement in beams. There are two main aspects which need to be considered: i) the boundary conditions in the specimen and ii) the heating scenario.

In the present work, the bond behavior was investigated on two specimen types: i) pull-out test specimen, which is widely used owing to relatively simple testing procedure, and ii) beam-end test specimens, which reflects the realistic bond behavior in beams. The two test specimens were first compared under ambient conditions. The pull-out specimens were further tested after exposure to slow heating for temperature range up to 700°C, whereas the beam-end test specimens were exposed to standard ISO 834 fire for duration of 30 minutes. The obtained results demonstrated a significant influence of the test specimen i.e. test method as well as the boundary and heating conditions on the bond behavior.

1 Introduction

Development of sufficient bond between reinforcement and concrete is necessary for the efficient transfer of forces from concrete to steel and vice-versa. Although bond strictly refers to the relative interfacial slip between the reinforcing bar and concrete, concrete splitting sometimes precedes the pull-out failure and the full bond strength cannot be achieved. Nevertheless, the bond strength is still calculated based on the surface area of the rebar and this is referred to as splitting bond strength. For instance, fib Model Code 2010¹ recommends an analytical bond stress-slip relationship (Figure 1) for pull-out failure or splitting failure under ambient conditions (room temperature) and monotonic loading as a function of different parameters such as concrete cover, concrete strength, rebar diameter etc. (Eq. 1).
Figure 1: Analytical bond stress-slip relationship for reinforcement in concrete under ambient conditions and monotonic loading.

\[
\tau_{u,\text{split}} = \eta_2 \cdot 6.5 \left( \frac{f_{cm}}{25} \right)^{0.25} \left( \frac{25}{\phi} \right)^{0.20} \left( \frac{c_{\min}}{\phi} \right)^{0.33} \left( \frac{c_{\max}}{c_{\min}} \right)^{0.10} + k_m K_{tr}
\]

where,

- \( \eta_2 \) factor to consider bond conditions [-]
- \( f_{cm} \) mean cylinder concrete compressive strength [MPa]
- \( \phi \) diameter of the anchored bar considered [mm]
- \( c_{\min} / c_{\max} \) minimum / maximum concrete cover
- \( k_m, K_{tr} \) factors to consider the influence of transverse reinforcement

If the concrete cover is small, for the rebar in low to medium concrete strength and relatively low confinement, concrete splitting limits the usable bond strength and the full bond strength cannot be activated. For the same concrete strength and level of confinement, increase in concrete cover leads to higher splitting resistance, and beyond a certain critical value of concrete cover, a pull-out failure takes place. The bond strength corresponding to pull-out failure defines the upper limit of the bond strength. Hence, any further increase in cover does not affect the bond strength. The model given by Eq. (1) and Figure 1 is valid for cast-in reinforcing bars in concrete at normal (room) temperature.

When subjected to elevated temperatures, the mechanical properties of concrete such as compressive strength, tensile strength, modulus of elasticity as well as bond strength between steel and concrete are significantly affected. Since the bond between steel and concrete can fail either due to bond pullout failure or concrete splitting failure, the degradation of the usable bond strength with temperature would also be different in case of the two failure modes. Further, it is also possible that the parameters for which a bond pullout failure would occur at ambient conditions could result in concrete splitting failure at elevated temperature. Currently no model exists to consider the degradation of usable bond strength between reinforcement and concrete after exposure to fire. The tests reported herein are carried out as an attempt to provide the required information towards development of one such model.
2 Literature review

The research on bond behavior after fire has so far mainly been focused on the investigations of pure bond failure using pullout specimens (pullout specimen according to RILEM\textsuperscript{2}), where the rebar is placed in the center of the specimen. The tests were in general performed under steady state conditions, i.e. the specimens were slowly heated to the target temperature and the residual pullout testing was performed upon cooling down to ambient temperature. The choice of the specimen (rebar in the center), heating scenario (slow heating), testing under confined conditions and the use of relatively small bonded lengths (2-5 times the rebar diameter) resulted in bond failure even at very high temperatures. There exist only very few studies aimed at investigating the thermal degradation of bond in case of concrete splitting, especially in case of realistic fire scenarios.

Morley and Royles\textsuperscript{2} studied the degradation of bond with temperature in normal strength concrete using a confined test setup. They performed tests on rebar in unstressed state (not loaded during heating) and in stressed state (loaded during heating). One of the major findings was the insignificant difference observed between the tests performed in stressed and unstressed state. Diederichs and Schneider\textsuperscript{3} performed a similar study on normal strength concrete using confined setup. They concluded that the reduction in bond strength with temperature follows the reduction in the compressive strength (rather than the tensile strength) of concrete. Similar conclusions were drawn based on studies conducted more recently by Haddad et al\textsuperscript{4}. Hertz\textsuperscript{5} found that the rebar diameter has little influence on the degradation of bond at temperatures up to 500°C. Bingöl and Gül\textsuperscript{6} reported significantly lower bond degradation for higher bonded length. Sager\textsuperscript{7} investigated the effect of relative rib area, aggregate type, concrete strength, concrete cover and the position of the rebar within the reinforced concrete element, being the first to differentiate between pure bond failure (pull-out of the rebar) and concrete splitting. The experiments demonstrated that the usable bond strength of a rebar is limited by concrete splitting failure, since it yields significantly lower pull-out loads than pure bond failure. In centric tests, the predominant failure mode was pull-out (bond failure), whereas all eccentrically loaded specimen failed due to concrete splitting.

Sharma et al.\textsuperscript{11} performed a numerical study to investigate the bond performance of reinforcement in concrete after exposure to fire loads using the beam-end test specimen. They highlighted the importance of considering realistic boundary conditions while testing the bond performance of the rebar subjected to fire loads. It was reported that even for relatively large covers where at ambient temperature a pure bond failure was observed, after exposure to fire the failure mode changed to concrete splitting. These results highlighted the crucial difference between influence of the state in which bond tests at elevated temperature are usually performed (slow heating, uniform cover and confined setup) and the state that may actually exist in reality (fast heating, non-uniform cover and unconfined setup). A test program is being pursued by the authors based on these findings.

Experimental results on thermal degradation of bond capacity available so far are useful for the development of temperature dependent material models for the interfacial zone between steel and concrete, but provide little information about the actual post-fire bond capacity in structural elements. In realistic fires initial heating rates are significantly higher than those in laboratory tests in electrical ovens. High temperature gradients in the concrete section, in particular in the case of relatively small concrete cover, result in severe damage (cracking) of the surface layer of concrete. In
such a case, the bond capacity in residual state can be limited by the local tensile capacity of concrete, i.e. concrete splitting precedes bond failure. In order to understand the post-fire bond behavior, it is hence necessary to i) differentiate between possible failure modes (pure bond failure of the interfacial zone between steel and concrete and concrete splitting) and ii) quantify the effect of the heating rate on the damage of the concrete cover, which can significantly reduce the bond capacity by promoting concrete splitting. In this work residual bond capacity measured in laboratory tests on standard pullout specimens exposed to slow heating is compared to the post-fire bond capacity determined on more realistic beam-end specimens exposed to standard ISO 834 fire. The test methods and results are presented in the following.

3 Experimental investigations

In this work the residual bond capacity was investigated using two types of experiments:

1. Standard pullout test specimens exposed to slow heating
2. Beam-end test specimens exposed to standard ISO 834 fire

3.1 Test specimens and materials

3.1.1 Pullout test specimen

Static pullout tests with confined test setup have been performed on reinforcement in concrete in the residual state after exposure to elevated temperature. Standard test rebar (grade BSt 500) with a diameter of $d_s = 16\text{mm}$ was placed in a prismatic concrete specimen with dimensions of 160mm x 160mm x 300mm, see Figure 2.

In order to prevent local concrete failure, the specimen was provided with a debonded bonded length of $7d_s (= 112\text{mm})$, followed by a bonded length of $8d_s (= 128\text{mm})$ and further followed by a debonded length of 60mm. The reinforcement cage consisted of 3 stirrups with a diameter of 8mm (one in the center of the bonded length and two at a distance of 100mm on both sided of the stirrup) and 4 longitudinal rebar with a diameter of 8mm. Tests were performed for two different positions of the rebar, namely, in the middle and at the edge of the specimen, see Figure 3. The debonding between steel and concrete was achieved by using glass tubes to cover the test rebar in debonded zones. In the case of the rebar at the edge the minimum concrete cover was $c_{\text{min}} = 1.5d_s = 24\text{mm}$.
Normal strength concrete of grade C16/20 with a maximum aggregate size of 8 mm was used in the tests. The average strength of concrete measured at an age of 28 days using the standard 150mm cubes was obtained as 23 MPa. The heating of the specimens to the target temperatures was performed at concrete age between 28 and 35 days.

3.1.2 Beam-end test specimen

Static pullout tests with unconfined test setup have been performed on reinforcement in concrete in the residual state after exposure to standard ISO 834 fire in a duration of 30 minutes. Standard test rebar (grade BSt 500) with a diameter of $d_r = 16$mm was placed in a beam end specimens of dimensions 300mm x 450mm x 640mm, see Figure 4.

Similar to the pullout specimen, the beam-end test specimen was provided with a debonded bonded length of $7d_r$ (= 112mm), followed by a bonded length of $8d_r$ (= 128mm) and further followed by a debonded length of 60mm. The diameter and distribution of the stirrups in the bonded zone corresponded to that in the pullout test specimens as well. Minimum concrete cover was $c_{min} = 1.5d_r = 24$mm, which corresponds to the cover of the pullout specimen with the rebar at the edge.
Normal strength concrete of grade C16/20 with a maximum aggregate size of 16 mm was used for casting the beam-end test specimens. The average strength of concrete measured at an age of 28 days was 21 MPa. The fire test was performed at concrete age of 75 days. The concrete compressive strength on the day of fire test was measured as 28.4 MPa.

3.2 Test program
In case of the pullout specimens the two above-mentioned two configurations, namely rebar positioned in the center and at the edge of the specimen, were tested. Each case was tested at room temperature and after exposure to elevated temperature of 300°C, 500°C and 700°C. For each case two specimens were tested. In case of the beam-end test specimens, the heating was performed according to ISO 834 fire for 30 minutes. A total of three specimens were tested. In order to provide reference values of bond capacity, three tests were performed under ambient conditions without prior exposure to fire.

3.3 Test procedure and setup

3.3.1 Heat treatment
In case of the pullout specimens, the test specimens were placed inside an electric oven. The target temperature in each case was reached at a relatively slow heating rate of 2°C/min and subsequently maintained for 3 hours to ensure uniform heating of the test specimen. The temperature was allowed to gradually come down after the completion of retention time by opening the small ventilation holes in the oven, while keeping the oven door closed. When the oven temperature reached approx. 150°C, the door of the oven was opened to allow relatively fast cooling.

In case of the beam-end tests, the specimens were placed in a fire furnace. The unexposed sides of the specimens were protected from heating using aerated concrete blocks and mineral wool. The heating was performed according to ISO 834 in a duration of 30 minutes. The heating was achieved using oil burners placed on either side of the test specimen. During the fire test, the oven temperature was measured and controlled using four plate-thermocouples placed in front of the heated side of the specimens approximately at the mid-height (two thermocouples on either side of the specimen). Upon finishing the heating phase, the ventilation ducts were opened and the specimens were allowed to cool down without opening the oven.

3.3.2 Pullout tests
On the pullout specimens, pullout tests with confined test setup were performed. The typical test setup for the pullout tests is shown in Figure 5. The specimen was placed with the rebar passing through the slot in the reaction bracket. A loading frame was used to connect the hydraulic cylinder to the test rebar, which was gripped using a wedge fixture. One LVDT was used to measure the rear end displacement (slip) of the test rebar and another LVDT was used to measure the crack width of the splitting crack, if any (Figure 5). A calibrated load cell was used to measure the applied load. The specimen was loaded with oil pressure control quasi-statically while continuously recording the applied load, the rear end displacement and the splitting crack width.
On the beam-end test specimens, pullout test with unconfined test setup were performed, see Figure 6. The connection of the test rebar to the hydraulic cylinder, displacement and load measurement was similar to that in the case of the pullout specimens.

4 Test results

4.1 Bond stress-slip curves

The applied load, $F$, was converted to average bond stress, $\tau_B$ over the bonded length, following the uniform bond stress approach:

$$\tau_B = \frac{F}{\pi d_s l_b} \quad (2)$$

Where $F$ is the applied tensile force on the rebar, $d_s$ is the diameter of the rebar and $l_b$ the bonded length ($l_b = 8 d_s$ in the present study).

4.1.1 Pullout specimens – rebar in the center

The results of the pullout tests performed on the specimens with test rebar located in the center are plotted in Figure 7. The curves display a typical bond stress-slip behavior in case of pure bond (pullout) failure. Initially, a linear ascending branch is seen followed by a nonlinear ascending branch showing relatively large increase in slip with small increase in the stress due to the crushing
of concrete in front of the lugs of the test rebar. This is followed by a plateau at the peak load until the concrete key between the lugs of the rebar is partially sheared off. Hereafter the bond stress gradually drops down until only frictional resistance is available. It is worth noting that the degradation of stiffness is observed only for temperatures above 300°C. Nevertheless, the general shape of all the curves is similar suggesting an occurrence of pullout failure in all the cases. This is also expected due to relatively large cover available on all sides of the test rebar, which would prevent premature splitting.

![Figure 7: Bond stress-slip curves obtained from pullout tests on specimens with rebar in the center](image)

4.1.2 Pullout specimens – rebar at the edge

The results of the pullout tests performed on the specimens with test rebar located at the edge are plotted in Figure 8.

![Figure 8: Bond stress-slip curves obtained from pullout tests on specimens with rebar at the edge](image)

In this case, the peak bond stress curves for a given temperature reach smaller peak stresses in comparison to those attained by the rebar in the center for the corresponding temperature, thus
indicating splitting failure of concrete before the pullout bond strength could be achieved. It is marked by the absence of plateau in the curves close to the peak stress.

4.1.3 Beam-end test specimens
The results of the pullout tests performed on the beam-end test specimens are plotted in Figure 9. Under ambient conditions, the failure mode and the absolute levels of the maximum bond stress correspond very well to those of the pullout specimens with rebar at the edge. Slightly higher bond stress in case of the beam-end test specimens can be attributed to higher concrete strength in beam-end test specimens. After 30 min of ISO 834 fire the residual bond capacity has reduced by approximately 55% accompanied by a marked degradation of stiffness.

![Figure 9: Bond stress-slip curves obtained from beam-end tests specimens](image)

4.2 Failure patterns

4.2.1 Pullout specimen
All the specimens were first checked for any cracks after exposure to elevated temperature. For the case of exposure temperature of 300°C, no cracks were visible on the surface of any of the specimen after the heat treatment. For the case of exposure temperatures of 500°C and 700°C, hairline cracks were visible on the surface, which were marked in red color. The cracks obtained as a result of the pullout test on the rebar were marked by blue color. In general, slow thermal exposure caused relatively moderate cracking, even at very high temperatures.

![Figure 10: Typical failure modes observed for the pullout specimens with rebar in the center](image)
All the specimens with the rebar located in the center failed by bond pullout without the formation of any splitting cracks, even at 700°C (Figure 10). This is attributed to relatively large concrete cover on all sides of the test rebar and is consistent with the shape of bond stress-slip curves obtained for this case (Figure 7). The specimens with the rebar located at the edge of the specimen displayed one major splitting crack parallel to the axis of the test rebar (Figure 11), which is formed due to the radial tensile stresses generated by pulling out the rebar.

4.2.2 Beam-end test specimen

Under ambient conditions the beam-end test specimens underwent splitting failure, see Figure 12. The tests on the beam-end test specimens displayed severe net cracking after the fire exposure (marked red in Figure 12), whereby the location of the larger cracks approximately coincided with the location of the stirrups within and close to the bonded length. When comparing these with those in Figure 11, it is evident that the higher heating rate in the ISO 834 fire resulted in a much greater extent of cracking. Consequently, the degradation of bond capacity is also much stronger in case of ISO 834 fire.

Thermally induced cracks in the bonded zone further widened during the residual pullout tests, but also new splitting cracks were formed (marked black in Figure 12).

4.3 Degradation of the bond capacity due to heating

The degradation of the peak bond stress versus maximum temperature is depicted in Figure 13. Already under ambient conditions a substantial decrease in the peak bond stress can be observed (for the pullout specimen) when failure mode changes from bond failure (rebar in the center) to splitting (rebar in the edge). If the specimen is exposed to slow heating, the degradation remains very low up to temperature of approximately 300°C. Thereafter almost a linear degradation of the bond strength
with temperature occurs. It is difficult to directly compare the results obtained on the slowly heated pullout specimens with those from the beam-end test specimens. In the former case, the temperature is constant throughout the section, whereas the latter case experiences very high thermal gradients across the section. One of the possible options is to compare the maximum temperature close to the test rebar in the beam-end test specimen to the maximum specimen temperature in the pullout specimen. The maximum temperature measured close to the rebar (27 mm from the surface) in the beam-end test specimen was approximately 290°C. If we compare the residual bond stress for the three cases, it becomes evident that the beam-end test specimen exhibits a residual bond capacity, which is less than half than that of the pullout specimen with the rebar in the center and approximately 40% lower residual bond capacity than the pullout specimen with the rebar at the edge. This level of degradation in case of the slow heating is reached only at a temperature of 500°C for the same concrete cover. These results confirm the assumption that the high heating rates promote concrete splitting and thus lead to a significantly more severe reduction of the residual bond capacity. For this reason, the fire performance cannot be estimated based on experiments performed with unrealistically low heating rates, since this approach could lead to unconservative results.

![Figure 13: Degradation of peak bond stress as a function of temperature for different rebar positions](image)

**5 Conclusions**

In this work, pullout tests were performed on two types of specimens exposed to two different heating scenarios. Confined tests have been performed on pullout specimens with reinforcement placed in the center and at the edge of the specimen, in order to investigate the effect of the failure mode on the effective bond capacity. The specimens were tested at room temperature and at 300°C, 500°C and 700°C, whereby the heating was performed at a very slow rate. In order to provide a comparison to a real fire scenario, beam-end test specimens with same concrete class, distribution of the reinforcement and bonded length were exposed to standard ISO 834 fire for duration of 30min.

In the case of pullout specimens a significant difference was observed between the specimen with the rebar in the center, which underwent pure bond failure over the complete temperature range, and the specimens with the rebar at the edge, which exhibited concrete splitting. The splitting failure occurred at significantly lower level of effective bond stress than the bond failure, but the relative
degradation with increasing temperature was comparative for both the specimens. In case of the beam-end test specimen exposed to standard fire a very strong degradation of the residual bond capacity was measured. This degradation is strongly influenced by the extent of thermally induced cracking, which is more prominent for higher heating rates. The beam-end test specimen exhibited significantly higher level of surface damage than any of the slowly heated specimens.

It can be concluded that the fire has a two-fold effect on the bond behavior: on one side it leads to a degradation of the material in the bond region and on the other side it results in concrete damage. The first effect depends primarily on the maximum temperature attained around the rebar and promotes pull-out failure, which is also captured in pullout tests with low heating rates. The second effect results from the stresses due to thermal gradients and promotes concrete splitting. Given the real fire scenarios and the configuration of structural members in real structures, the second effect is generally more dominant than the first effect. Hence, it is necessary to consider both the aspects (failure mode and heating rate) when considering the post-fire bond capacity.

Further research is required in order to quantify the effects of heating rate on the bond behavior for various concrete cover levels and rebar diameters. It is in particular interesting to investigate under which conditions the pullout failure under ambient conditions can change to splitting failure after fire exposure. The authors are currently pursuing an experimental and numerical study on this topic.

References:

1. fib. Model Code for Concrete Structures 2010. vol. 55. Ernst & Sohn; 2013


ANALYSIS OF FACTORS INFLUENCING BOND BEHAVIOUR BETWEEN REINFORCING STEEL BARS AND FRC UNDER MONOTONIC LOADING

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ABSTRACT

Bond phenomenon allows the variation of the bar tensile action to be transferred to the surrounding concrete by means of shear stresses developing along the lateral surface of the bar. This mechanism entails the formation of radial transversal pressure as a result of the wedge action of the crashed concrete between the ribs. The latter may cause the onset of longitudinally splitting cracks when the tensile strength is reached in the cover. This phenomenon depends mainly on the post-peak behavior of concrete, which can be significantly enhanced with the addition of fibers. Based on the available results from the literature, in the first part of the paper an analysis on the bond behavior between steel rebars and FRC is presented. The influence of several parameters, such as concrete strength, fibers content, aspect ratio and concrete cover is evaluated. Finally, the preliminary results of pull-out tests on short anchorages will be also presented under monotonic loading, embedded in plain concrete and fiber reinforced concrete. The test set up was able to simulate the influence zone of one stirrup. A volume content of steel hooked fibers equal to 0.38% has been investigated. The test results shows the addition of steel hooked steel fibers increases bond strength and reduces splitting crack opening in conventional FRC.

1 Introduction

Bond behavior mainly depends on the mechanical interlocking between concrete and bar ribs. This mechanism involves the formation of radial transversal pressure as a result of the wedge action of the crashed concrete between the ribs¹,². The latter may cause the onset of longitudinally splitting cracks when the tensile strength is reached in the concrete cover³,⁴. The propagation of splitting cracks and thus the brittle failure of the anchorage or a splice can be efficiently counteracted by the confinement action provided by transversal reinforcements⁵, by transversal pressure⁶,⁷ or by cohesive stresses between the surfaces of splitting cracks⁴. This last contribution depends mainly on the post-peak behavior of concrete⁸, which can be significantly enhanced by the addition of fibers to the concrete mixture.

In the available literature has been widely verified that adding fibers to concrete matrix improves most of the mechanical properties of the material such as tensile strength, energy absorption and
toughness as well as fatigue strength. The presence of fibers in concrete has also a positive effect on the bond of reinforcing steel bars. Even though the bond behavior of steel bars in concrete is controlled by several phenomena (such as chemical adhesion, frictional resistance, mechanical interlock), the presence of fibers improves the bond properties by means of the post-peak toughness of FRC in tension due to bridging action on the splitting cracks (Figure 1)\textsuperscript{5}. Thus, at the onset of cracking fibers contribute to limit cracks opening and to resist additional tensile forces which the concrete itself cannot sustain\textsuperscript{9}.

![Figure 1: Effect of fibers on bond](image)

In has been also shown that the addition of fibers to concrete matrix improves the bond-slip behavior of the reinforcing steel bars after the development of splitting cracks leading to more ductile failures\textsuperscript{10,11}. Two different modes of failure can be observed, depending on cover and confinement: pull-out failure and splitting failure. In the former case, bond failure is mostly due to the shearing off of the concrete keys cast between each pair of lugs. In the latter case, bond failure is mostly due to the longitudinal splitting of the concrete surrounding the bar. Many authors concluded that fibers improve bond capacity in terms of ductility\textsuperscript{12-15} while their effect on the bond strength could be appreciated when the loss of bond is due to splitting\textsuperscript{16,17}.

In this paper an analysis of the available literature on the bond of steel reinforcement in fiber reinforced concrete is presented in order to better understand the most important parameters which affect the bond behavior. Moreover, this analysis is an effective tool for the fib-TG 2.5 “Bond and Material Models” (http://fibt25bond.unibs.it) which aims at developing a simplified analytical/physical model of bond in fiber reinforced concrete. Finally, the first results of pull-out tests on short embedded length in conventional FRC shows the effectiveness of steel fibers to improve bond behavior in term of strength, maximum slip and splitting crack opening.
2 A database from literature survey

A preliminary analysis has been carried out considering several parameters, such as type of bond (lapped splice, short embedded length, long embedded length), type of investigation (experimental or analytical), type of concrete matrix and type of fiber. Afterwards, considering the predominance of papers on concretes reinforced with steel fibers, the analysis has been focused on the bond in SFRC\(^{15, 17-46}\). As shown in Figure 2, the characteristics parameters are fiber content and \(l/d_f\) ratio (where \(l\) is the fiber length and \(d_f\) is its nominal diameter).

On the basis of the available bond experimental data described in the selected papers, a more detailed analysis has been carried out considering a total number of 448 specimens, with 184 failure by splitting (41\%) and 264 by pull-out (59\%)\(^{18-30, 46, 47}\). The ratio \((c/d_b)\) between the concrete cover and the bar diameter may govern the failure mode (splitting or pull-out). With a \(c/d_b \leq 2.5\) the main failure mode is by splitting (82\%), while 96\% of pull-out failures has been recorded with \(c/d_b > 2.5\). Figure 4 shows that the effectiveness of fibers varies with the concrete mechanical properties and the average bond maximum stress is obtained for compressive strength ranging between 40 and 50 MPa.

![Figure 2: Main variables in the available literature](image1)

![Figure 3: Effect of the \(c/d_b\) ratio on failure mode](image2)
The effect of fiber content ($V_f$) and $c/db$ ratio on the normalized bond stress has been evaluated for both failure modes. As shown in Figure 5, it cannot be noticed a distinguishing trend. When pull-out failure is considered (Fig. 5a), for a given concrete cover the normalized bond strength increases with the volume content of fibers, even though this trend is not clearly noticeable for conventional concrete ($V_f<0.7\%$). The tests characterized by splitting failure seem to show an opposite trend, with the normalized bond strength decreasing with the increase of volume content of fibers. Therefore, the benefit of bond from the capability of FRC post-cracking residual strength to enhance the splitting strength of bond is not evident. More experimental studies are needed in conventional FRC with the aim to investigate the effect of the FRC casting direction on the orientation of the fibers crossing the potential splitting surfaces. In fact, any of many published papers on bond in FRC since '90s does not consider phenomenon when the test set up was design or when the test results were commented.

As proposed by some researches$^{17}$, the normalized bond strength is plotted as a function of the parameter $[(V_f L_b/d_b)/(c/db)]$ which takes into account amount of fibers in the concrete cover. It should be observed that a linear trend both for short ($L_b/d_b<5$) and longer embedded length ($5\leq L_b/d_b<10$), in accordance with the semi-empirical models proposed by Harajli$^{40}$ if pull-out failures are considered. The scatter of the experimental data is very wide if splitting failure is considered since the splitting resistance can be affected by the casting direction and so by the orientation of fibers in relation to the splitting surface.
Finally, a comparison in terms of maximum bond stress between experimental data from the considered papers and predicted data according to the local bond-slip relationship provided in the Model Code48 has been performed. As shown in Figure 7, for pull-out failure modes there is more experimental data dispersion, while in the case of splitting failure experimental data are in general higher than the predicted ones.

3 Experimental program

3.1 Materials, specimen geometry and test set-up

In the present paper, the results from pull-out tests on short anchorages, simulating the influence zone $\Delta z$ of one stirrup, are presented. Tests are performed on specimens made with normal as well as high-strength concrete, with and without steel fibers. Furthermore, the influence of the transverse reinforcement area was investigated by testing specimens with different stirrup diameters.

The specimens simulate a concrete block with one anchored and one two-legged stirrup (Fig. 8). The concrete block embedding the bar is somewhat simplified being cast as a thick plate with one bar lying in the mean plane AA. The anchored bars had a diameter ($d_b$) of 20 and 24 mm, and embedded lengths ($\Delta z$) of 150 mm and 200 mm respectively ($\Delta z/d_b \geq 8.5$). The stirrups had different diameters $d_{st}$ with the purpose of studying bond phenomena for different amounts of transverse reinforcement; in order to measure the confining contribution of the concrete cover and the concrete spacing between
bars, some specimens had no stirrups. The geometrical characteristics of the specimens are shown in Tab. 1, where $\Omega$ is the Stirrup Index of Confinement, which is defined as the ratio between the global cross-section area $A_{st} (= n_{st} \pi d_{st}^2 / 4)$ of the stirrup legs and the area of the principal bar in the splitting plane $A^* (= n_b d_b \Delta z)$, and $B$ is the Concrete Index of Confinement, which is defined as the ratio between the net area of the concrete $=((b - n_b d_b) \Delta z)$ in the splitting plane and the area $A^*$ (Giuriani et al., 1991). These two parameters govern bond behavior and are relevant to structural design; a parameter similar to $\Omega$ has been recently introduced in Model Code 2010 and in ACI 318 code (from 1995).

Figure 8: Specimen geometry and instrument placement

The instrumentation consisted of a set of LVDTs to measure splitting crack opening close to the stirrup as well as the bar-to-concrete slip at both loaded and unloaded ends of the bar (Fig. 8). The splitting crack opening close to the stirrups was measured by two LVDTs, one on each side of the specimen. In order to eliminate the confining action due to the friction between the specimen and the reaction plate, the experimental set-up proposed by Plizzari et al. was adopted. The specimen was assembled in an Instron 2714/8500 test machine. The signal from the LVDT of the actuator was used as the control parameter. The load applied by the machine was measured by a reversible load cell with a capacity of 250 kN of the Instron.

Four different mixes were prepared: one reference mix of normal-strength concrete (NC), one mix with the same matrix and the addition of steel fibers (NSC), one mix of high-strength concrete (HC) while the last mix had the same high-strength concrete matrix with the addition of steel fibers (HSC). The maximum aggregate size was 15 mm. Melamine plasticizer and acrylic based superplasticizer were adopted for normal and high-strength concrete respectively. The hooked steel fibers were low carbon, cold drawn, 30 mm long and had a diameter of 0.5 mm (aspect ratio=60), an elastic modulus of 210,000 MPa and a tensile strength higher than 1100 MPa. The volume fraction of steel fibers was 0.38% for both normal and high-strength concrete. It should be observed that the specimens were casted with the anchored bar placed vertically, thus allowing the fibers to be oriented perpendicularly
to the potential splitting surface. Table 2 shows the concrete mix proportions, the properties of fresh concrete as well as the compression strength \( (f_c) \), the direct tensile strength \( (f_{ct}) \) and the elastic modulus of elasticity \( (E_c) \), all measured from cylinders. Anchored bars and stirrups were made of B500B steel, usually employed in Europe, having a nominal yielding strength of 500 MPa.

Table 1: Geometrical characteristics, bond strength \( (\tau_{max}) \) and splitting crack width at peak load \( (w_p) \).

<table>
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<tr>
<th>Material</th>
<th>( d_b ) [mm]</th>
<th>( d_{st} ) [mm]</th>
<th>( b ) [mm]</th>
<th>( \Delta z ) [mm]</th>
<th>( \Omega ) [-]</th>
<th>( \tau_{max} ) [MPa]</th>
<th>( w_p ) [mm]</th>
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<tr>
<td></td>
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<tr>
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<td>9.6</td>
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<td></td>
<td>24</td>
<td>6</td>
<td>120</td>
<td>200</td>
<td>0.012</td>
<td>10.8</td>
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<tr>
<td></td>
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<td>100</td>
<td>175</td>
<td>0.016</td>
<td>13.3</td>
<td>0.33</td>
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<tr>
<td></td>
<td>20</td>
<td>6</td>
<td>100</td>
<td>175</td>
<td>0.016</td>
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<tr>
<td></td>
<td>24</td>
<td>8</td>
<td>120</td>
<td>200</td>
<td>0.021</td>
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<td>8</td>
<td>120</td>
<td>200</td>
<td>0.021</td>
<td>11.6</td>
<td>0.18</td>
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<tr>
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<td>200</td>
<td>0.033</td>
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<td>High Strength FRC (HC) ( V_f=0% )</td>
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<td>7.7</td>
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<td>0.016</td>
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<td>15.5</td>
<td>0.140</td>
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<td></td>
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<td>100</td>
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<td>0.016</td>
<td>15.9</td>
<td>0.150</td>
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<td>0.021</td>
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### Table 2: Mix-design, fresh-state and mechanical properties of concrete.

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<th>Concrete Type</th>
<th>Cement Type</th>
<th>Cement [kg/m³]</th>
<th>Silica f. [kg/m³]</th>
<th>w/c ratio</th>
<th>Superp. [kg/m³]</th>
<th>Slump [mm]</th>
<th>f_c [MPa]</th>
<th>f_ct [MPa]</th>
<th>E_c [MPa]</th>
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<td>NS</td>
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<td>-</td>
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<td>120</td>
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<td>180</td>
<td>85</td>
<td>5.7</td>
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</tbody>
</table>

### 3.2 Test results

In all specimens, owing to the presence of the preformed crack between the steel angles at the loaded end of the specimen, and to the orientation of the bar ribs, a main splitting crack formed along plane AA of Fig. 8. In some specimens, secondary splitting cracks appeared in other longitudinal planes (generally after reaching the peak load). The values of maximum bond stress ($\tau_{\text{max}}$, assumed evenly distributed along the bar) and of splitting crack opening at maximum load ($w_p$) are shown in Table 1. Figure 9a exhibits the bond stress as a function of the loaded end slip, as obtained from tests on specimens without stirrups. The remarkable increment of stiffness and bond strength in specimens with fibers can be noticed. These specimens are usually characterized by a very brittle behavior\(^5\); however, when fibers are adopted, a more ductile behavior is obtained. In fact, the ultimate loaded end slip of fiber-reinforced concrete is three times the value observed from concrete without fibers. This underlines the importance of the confining action of the cohesive stresses transmitted across the concrete splitting surface, and the remarkable influence of fibers on anchorage behavior. Figure 9b concerns specimens with two transverse bars having a diameter of 8 mm. Again, fibers increase bond stiffness and strength; the bars anchored in high-strength concrete with fibers yielded before reaching the maximum bond strength, although a very short embedded length was adopted.

![Figure 9: Bond stress versus loaded end slip as obtained from specimens without stirrups (a) and from specimens with stirrups having a diameter of 8 mm (b)](image_url)

Figure 10a shows the average maximum bond stress $\tau_{\text{max}}$ versus the stirrup index of confinement, as obtained from all specimens tested. In specimens without transverse reinforcement ($\Omega=0$), the confining action is due only to the contribution of the concrete surrounding the bars. The results from specimens (of the same material) having the same concrete index of confinement ($B$) are connected by a continuous line. The larger the stirrup index of confinement, the higher the bond capacity becomes. As mentioned above, fibers increase bond strength and have a better efficiency in the high-
strength matrix. The splitting crack opening decreases when a higher stirrup index of confinement is adopted, and is larger in high-strength concrete because of its more brittle behavior (Fig. 10b). Steel fibers reduce the splitting crack opening, thus improving concrete durability.

Figure 10: Bond capacity (a) and splitting crack opening at maximum load (b) versus Stirrup Index of Confinement

4 Concluding remarks

The analysis of the experimental results published in literature shows that the bond of steel ribbed bars in FRC can remarkably benefit from the post-peak behavior of FRC, particularly when the bond is governed by pull-out failure. However, there is a wide scatter of results when the anchorage fails by concrete splitting thus showing that further research is needed when conventional FRC is used ($V_f < 0.5\%$). The paper presents also the results of pull-out tests on short anchorages of ribbed bars in normal and high-strength concrete. Although a low volume fraction of fibers was adopted, the experimental results show that hooked steel fibers make anchorage behavior of rebars without transverse reinforcement much more ductile, and increase bond stiffness with respect to concrete without fibers. Bond capacity increases with stirrup index of confinement and with concrete strength. Splitting crack opening is larger in high-strength concrete because of its more brittle behavior. Hooked steel fibers increase bond strength and reduce splitting crack opening; they are more effective in a high-strength concrete matrix.

5 Acknowledgement

The experimental research was supported by the University of Brescia. The know-how and the technical expertise of the technicians of the Laboratory P. Pisa of the University of Brescia is gratefully acknowledged.

References:


49. ACI Committee 318, 2014, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary, American Concrete Institute, Farmington Hills, Mich.
The bond performance of reinforcement in concrete at elevated temperatures is often investigated using a pullout test setup with the rebar positioned in the center of the test specimen. Based on these tests, significant data is available in the literature that displays a relatively gradual degradation of the bond strength as a function of temperature. However, these tests often do not consider a highly important aspect of the bond performance that is concrete splitting. In this work, the bond performance between reinforcement and concrete is experimentally investigated after exposure to elevated temperature. Cuboidal test specimens (square prisms of size 160mm x 160mm x 300mm) made of normal strength concrete with test rebar of ds = 16mm diameter and confining reinforcement using 8mm diameter stirrups are used for these tests. The test rebar is provided with a debonded length of 112mm (7ds) in the front, followed by a bonded length of 128mm (8ds) and further followed by another debonded length of 60mm inside the specimen. Three different rebar positions are investigated in this program, namely, rebar located in the center of the specimen (clear cover to both edges, cmin = cmax = 72mm), rebar at the edge (clear cover to closer edge, cmin = 24mm and clear cover to other edge, cmax = 72mm) and rebar at the corner (clear cover to both edges, cmin = cmax = 24mm). The tests are performed by first exposing to the test specimens to slow heating until reaching a steady state for different levels of target temperature, then cooling down to room temperature and later followed by pullout testing using a confined test specimen. Through the pullout test, the applied load and the corresponding rear-end slip of the rebar is measured.

The test results display that the degradation of the usable bond strength of the rebar with elevated temperature is dependent on the failure mode. In case of rebar located away from the edges, bond failure dominates and the degradation with elevated temperature is rather gradual. However, in case of rebar close to an edge or at corner, concrete splitting dominates leading to a steeper degradation in the usable bond strength as a function of the temperature.

1 Introduction

Development of sufficient bond between reinforcement and concrete is necessary for the efficient transfer of forces from concrete to steel and vice-versa. Although bond strictly refers to the relative interfacial slip between the reinforcing bar and concrete, concrete splitting sometimes precedes the pull-out failure and the full bond strength cannot be achieved. Nevertheless, the bond strength is still...
calculated based on the surface area of the rebar and this is referred to as splitting bond strength. For instance, fib Model Code 2010 recommends an analytical bond stress-slip relationship (Figure 1) for pull-out failure or splitting failure under ambient conditions (room temperature) and monotonic loading as a function of different parameters such as concrete cover, concrete strength, rebar diameter etc. (Eq. 1).

\[
\tau_{bu,split} = \eta_2 \cdot 6.5 \left( \frac{f_{cm}}{25} \right)^{0.25} \left( \frac{25}{\phi} \right)^{0.20} \left[ \left( \frac{c_{min}}{c} \right)^{0.33} \left( \frac{c_{max}}{c_{min}} \right)^{0.10} + k_m K_{tr} \right]
\]  

(1)

where,
- \( \eta_2 \) factor to consider bond conditions [-]
- \( f_{cm} \) mean cylinder concrete compressive strength [MPa]
- \( \phi \) diameter of the anchored bar considered [mm]
- \( c_{min} / c_{max} \) minimum / maximum concrete cover
- \( k_m, K_{tr} \) factors to consider the influence of transverse reinforcement

If the concrete cover is small, for the rebar in low to medium concrete strength and relatively low confinement, concrete splitting limits the usable bond strength and the full bond strength cannot be activated. For the same concrete strength and level of confinement, increase in concrete cover leads to higher splitting resistance, and beyond a certain critical value of concrete cover, a pull-out failure takes place. The bond strength corresponding to pull-out failure defines the upper limit of the bond strength. Hence, any further increase in cover does not affect the bond strength. The model given by Eq. (1) and Figure 1 is valid for cast-in reinforcing bars in concrete at normal (room) temperature.

When subjected to elevated temperatures, the mechanical properties of concrete such as compressive strength, tensile strength, modulus of elasticity as well as bond strength between steel and concrete are significantly affected. Since the bond between steel and concrete can fail either due to bond pullout failure or concrete splitting failure, the degradation of the usable bond strength with temperature would also be different in case of the two failure modes. Further, it is also possible that the parameters for which a bond pullout failure would occur at ambient conditions could result in
concrete splitting failure at elevated temperature. Currently no model exists to consider the degradation of usable bond strength between reinforcement and concrete after exposure to elevated temperature considering different position of the rebar (middle, edge or corner). The tests reported herein are carried out as an attempt to provide the required information towards development of one such model.

2 Literature review

Significant research has been performed so far to investigate the influence of elevated temperature on bond characteristics between steel reinforcement and concrete in case of bond pullout failure. The experiments were performed mainly using a pullout test setup and confined conditions. However, the information on the bond performance of the reinforcement in concrete after exposure to elevated temperature in case of concrete splitting failure is rather limited.

Morley and Royles² studied the degradation of bond with temperature in normal strength concrete using a confined test setup. They performed tests on rebar in unstressed state (not loaded during heating) and in stressed state (loaded during heating). Furthermore, pull-out tests were performed in hot state and residual state. One of the major findings was the insignificant difference observed between the tests performed in stressed and unstressed state. The specimens loaded in hot state exhibited slightly higher bond strength for temperatures above 500°C in comparison with the corresponding tests performed in residual state. Diederichs and Schneider³ performed a similar study on normal strength concrete using confined setup. They concluded that the reduction in bond strength with temperature follows the reduction in the compressive strength (rather than the tensile strength) of concrete. Similar conclusions were drawn based on studies conducted more recently by Haddad et al.⁴. Hertz⁵ found that the rebar diameter has little influence on the degradation of bond at temperatures up to 500°C. Bingöl and Gül⁶ studied the influence of embedment depth (bonded length) on degradation of bond strength with temperature. They reported significantly lower bond degradation for higher bonded length.

Sager⁷ investigated the effect of relative rib area, aggregate type, concrete strength, concrete cover and the position of the rebar within the reinforced concrete element being the first to differentiate between pure bond failure (pull-out of the rebar) and concrete splitting. The experiments demonstrated that the usable bond strength of a rebar is limited by concrete splitting failure, since it yields significantly lower pull-out loads than pure bond failure. In centric tests, the predominant failure mode was pull-out (bond failure), whereas all eccentrically loaded specimen failed due to concrete splitting.

Xiao et al.⁸ investigated the bond degradation after exposure to elevated temperatures using the RILEM beam specimen⁹. The study was performed for high strength concrete (f_c = 95 MPa) heated at approx. 15°C/min (retention time at target temperature 2 hours). Up to 400°C the bond degradation was very moderate, hereafter a sharp decrease in residual bond was observed. Lo Monte and Gambarova¹⁰ carried out investigations on reinforced concrete members to study the corner spalling and tension stiffening (compatibility side of bond) after exposure to fire. In half of the tests, the damage due to fire on bond was so high that the tension stiffening could not be tested. However,
they reported that in case of limited spalling, post-fire tension stiffening may still be effective and this aspect should be considered while planning a repair activity.

Relatively recently, Sharma et al.\textsuperscript{11} performed a numerical study to investigate the bond performance of reinforcement in concrete after exposure to fire loads using the beam-end-test specimen. They highlighted the importance of considering realistic boundary conditions while testing the bond performance of the rebar subjected to fire loads. It was reported that even for relatively large covers where at ambient temperature a pure bond failure was observed, after exposure to fire the failure mode changed to concrete splitting. These results highlighted the crucial difference between influence of the state in which bond tests at elevated temperature are usually performed (slow heating, uniform cover and confined setup) and the state that may actually exist in reality (fast heating, non-uniform cover and unconfined setup). A test program is being pursued by the authors based on these findings. However, a missing link remains to be understood, namely the influence of temperature on the bond strength of rebar in concrete with reference to the position of the rebar e.g. away from the edge, at the edge or at the corner. This work is a step forward to fill this gap in the current know-how.

3 Experimental investigations

3.1 Test specimen

In this work, static pullout tests with confined test setup have been performed on reinforcement in concrete in the residual state after exposure to elevated temperature. Square prismatic (Cuboid) concrete specimens of dimensions 160mm x 160mm x 300mm are used to carry out the tests (refer to Figure 2). Standard test rebar (European grade BSt 500) of diameter, $d_s = 16$mm are used to perform the tests. The test rebar is provided with a debonded length of $7d_s (= 112$mm) in the front, followed by a bonded length of $8d_s (= 128$mm) and further followed by a debonded length of 60mm. For practical reasons of gripping the test rebar and measuring the rear end slip, the test rebar protruded out of the test specimen by 150mm in the front and 50mm in the rear.

![Figure 2: Cross section of the pullout test specimen](image)

The test specimen was provided with three rectangular stirrups of diameter 8mm, one of which was located at the center of the bonded length while the other two stirrups were placed at a distance of...
100mm on either side of this stirrup (see Figure 2). To keep the stirrups in position, 8mm diameter rebar were placed at the corners of the specimen to form a reinforcement cage. Three different positions of the rebar were investigated (Figure 3) namely, in the middle of the specimen, at the edge and at the corner. For testing at the edge and corner, the minimum concrete cover was kept as $c_{\text{min}} = 24\text{mm} (= 1.5d_s)$.

Figure 3: Positions of test rebar investigated in the test program

The debonded zones were made by using glass tubes covering the test rebar for the specified length. The glass tubes were selected instead of plastic tubes as they could melt at high temperatures. Figure 4 shows a typical rebar sample prepared for casting.

Figure 4: Prepared test rebar for casting

3.2 Material properties

Normal strength concrete of grade C16/20 with a maximum aggregate size of 8 mm was used in the tests. Concrete mix is presented in Table 1.

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<thead>
<tr>
<th>Mix component</th>
<th>Quantity [kg/m$^3$]</th>
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</tr>
<tr>
<td>Powdered limestone</td>
<td>75</td>
</tr>
<tr>
<td>Sand (0/2)</td>
<td>816</td>
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<tr>
<td>Aggregate (2/4)</td>
<td>598</td>
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<tr>
<td>Aggregate (4/8)</td>
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<tr>
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</table>

The binder was chosen as a blend of cement and powdered limestone to achieve desired target compressive strength while assuring sufficient workability. A small dosage of superplasticizer was added to the concrete mix to optimize the workability. The average strength of concrete measured at
an age of 28 days using the standard 150mm cubes was obtained as 23 MPa. The heating of the specimens to the target temperatures was performed at concrete age between 28 and 35 days.

3.3 Test program
The test program consisted of testing the above-mentioned three configurations, namely rebar positioned in the middle of the specimen, rebar at the edge and rebar at the corner. Each case was tested at room temperature and after exposure to an elevated temperature of 300°C, 500°C and 700°C. For each case two specimens were tested to verify repeatability. The test program is summarized in Table 2.

Table 2: Test program for bond behavior of reinforcement in concrete under elevated temperature

<table>
<thead>
<tr>
<th>Rebar position</th>
<th>Target temperature</th>
<th>Center</th>
<th>Edge</th>
<th>Corner</th>
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<td></td>
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<td>2</td>
<td>2</td>
</tr>
<tr>
<td>300°C</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>500°C</td>
<td>2</td>
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<td>2</td>
</tr>
<tr>
<td>700°C</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
</tbody>
</table>

3.4 Test procedure and setup

3.4.1 Heat treatment
For heating, the test specimens were placed inside an electric oven with programmable controls for the heating rate as well as retention time for the temperature. The target temperature in each case was reached at a relatively slow heating rate of 2°C/min. After reaching the desired temperature, the temperature was maintained for 3 hours to ensure uniform heating of the test specimen. The temperature was allowed to gradually come down after the completion of retention time by opening the small ventilation holes in the oven, while keeping the oven door closed. When the oven temperature reached approx. 150°C, the door of the oven was opened to allow relatively fast cooling.

3.4.2 Pullout tests
On the heat treated specimens, pullout tests with confined test setup were performed. The typical test setup for the pullout tests is shown in Figure 5.

![Test setup employed for performing pullout tests](image)
The specimen was placed with the rebar passing through the slot in the reaction bracket. A loading frame was used to connect the hydraulic cylinder to the test rebar, which was gripped using a wedge fixture. One LVDT was used to measure the rear end displacement (slip) of the test rebar and another LVDT was used to measure the crack width of the splitting crack, if any (Figure 5). A calibrated load cell was used to measure the applied load. The specimen was loaded with oil pressure control quasi-statically while continuously recording the applied load, the rear end displacement and the splitting crack width.

4 Test results

4.1 Bond stress-slip curves

The applied load, \( F \), was converted to average bond stress, \( \tau_b \) over the bonded length, following the uniform bond stress approach:

\[
\tau_b = \frac{F}{\pi d_s l_b}
\]

(2)

Where,

- \( F \) = applied tensile force on the rebar
- \( d_s \) = diameter of the rebar
- \( l_b \) = bonded length = 8 \( d_s \) (in the present study)

4.1.1 Specimen with rebar in the center

The results of the pullout tests performed on the specimens with test rebar located in the center are plotted in Figure 6. The curves display a typical bond stress-slip behavior in case of pullout failure. Initially, a linear ascending branch is seen followed by a nonlinear ascending branch showing relatively large increase in slip with small increase in the stress due to the crushing of concrete in front of the lugs of the test rebar. This is followed by a plateau at the peak stress until the concrete key between the lugs of the rebar is partially sheared off. Further slippage of the rebar requires lesser force and the bond stress gradually drops down until only frictional resistance is available, which is marked by the flat region of the curve at large displacements.

It is worth noting that up to a temperature of 300°C, the bond stress-slip curves show almost a similar value of stiffness as in case of the reference curves (20°C), with a small reduction in the peak bond stress. For higher temperatures, a substantial reduction in the stiffness as well as in the peak bond stress is observed. Nevertheless, the general shape of all the curves is similar suggesting an occurrence of pullout failure in all the cases. This is also expected due to relatively large cover available on all sides of the test rebar, which would prevent premature splitting.

4.1.2 Specimen with rebar at the edge

The results of the pullout tests performed on the specimens with test rebar located at the edge are plotted in Figure 7. In this case, the bond stress-slip curves for a given temperature reach smaller peak stresses in comparison to those attained by the rebar in the center for the corresponding temperature. This suggests the occurrence of premature splitting of the concrete cover prior to reaching the pullout bond strength, under the influence of tensile stresses radiating out of the test rebar.
rebar into the surrounding concrete with small cover. It is marked by the absence of plateau in the bond stress-slip curves close to the peak bond stress. All the curves display a relatively pointed peak followed by a steeper descending branch compared to the curves shown in Figure 6.

Figure 6: Bond stress-slip curves obtained from pullout tests on specimens with rebar in the center

![Figure 6](image)

Figure 7: Bond stress-slip curves obtained from pullout tests on specimens with rebar at the edge

![Figure 7](image)

4.1.3 Specimen with rebar in the corner

The results of the pullout tests performed on the specimens with test rebar located in the corner are plotted in Figure 8. Again, the bond stress-slip curves are marked with sharp peak and steep descending branch associated with concrete splitting failure. Due to the rebar positioned at the corner it is surrounded by small cover on two sides, which led to even smaller peak bond stresses reached compared to the rebar at positioned at the edge. It is interesting to see that the stiffness remains almost unchanged until 500°C but shows a significant reduction for 700°C.
4.2 Failure patterns

All the specimens were first checked for any cracks after exposure to elevated temperature. For the case of exposure temperature of 300°C, no cracks were visible on the surface of any of the specimen after the heat treatment. For the case of exposure temperatures of 500°C and 700°C, hairline cracks were visible on the surface, which were marked in red color. The cracks obtained as a result of the pullout test on the rebar were marked by blue color.

4.2.1 Specimen with rebar in the center

All the specimens with the rebar located in the center failed by bond pullout without the formation of any splitting cracks (Figure 9).

![Figure 9: Typical failure modes observed for the specimens with rebar in the center](image)

Even at high temperatures, where the surface cracks were visible on the concrete after exposure to temperature, the failure in the pullout tests was due to bond failure. This is attributed to relatively large concrete cover on all sides of the test rebar and is consistent with the shape of bond stress-slip curves obtained for this case (Figure 6).

4.2.2 Specimen with rebar at the edge

The specimens with the rebar located at the edge of the specimen displayed one major splitting crack parallel to the axis of the test rebar (Figure 10), which is formed due to the radial tensile stresses generated by pulling out the rebar.
4.2.3 Specimen with rebar in the corner

The tests on the specimens with the rebar in the corner displayed severe splitting along the length of the test rebar, sometimes on both faces of the specimen close to the rebar. Along with the major longitudinal splitting crack, certain minor branching splitting cracks were also observed (Figure 11).

4.3 Bond strength vs. temperature

The degradation of the peak bond stress versus maximum temperature is depicted in Figure 12. Already under ambient conditions a substantial decrease in the peak bond stress can be observed when failure mode changes from bond failure (rebar in the center) to splitting (rebar in the edge and at the corner). In case of the rebar in the corner the bond stress reduces by half compared to the bond stress corresponding to the bond failure.
Up to temperatures of approximately 300°C the degradation of bond capacity is low to moderate. At these temperature levels no visible cracking of the outer surface could be observed. Beyond this temperature the bond capacity drops sharply, since both relevant material properties, compressive and tensile strength, degrade significantly at higher temperatures. The strong degree of material degradation is also visible in the crack patterns obtained due to heating, see Figures 9-11. In case of the rebar in the center, the shape of the curve is very similar to the degradation of the compressive strength, see for example fib Model Code 2010. As the splitting becomes dominant, the shape of the curve in Figure 12 follows rather the degradation curve for tensile strength, which decreases approximately linearly with increasing temperature.

5 Conclusions

In this work, pullout tests with confined test setup have been performed on reinforcement in concrete after exposure to elevated temperature (20°C to 700°C). In order to study the effect of the failure mode on the usable bond capacity, three different configurations of the rebar were investigated (rebar in the center, at edge and in the corner of the concrete specimen). The first configuration yielded pure bond failure (pullout) for the complete temperature range, whereas a splitting failure was observed for the latter two configurations. As can be expected, the splitting failure occurred at significantly lower level of usable bond stress than the bond failure and the usable bond stress reduced with decreasing concrete cover.

The results presented herein underline the need to consider the failure mode when investigating the effect of elevated temperatures on bond behavior. In the case of bond failure, the thermal degradation is gradual and approximately corresponds to thermal degradation of compressive strength. This result correlates well to the results found in the literature. In case of the splitting failure, however, there is a more pronounced degradation with temperature, which is approximately linear with increasing temperature. This degradation follows the thermal degradation of the tensile strength rather than that of the compressive strength.

It was also found that the presence of thermal cracks significantly influences the residual capacity. Hence, the choice of the heating scenario (slower or faster heating) is expected to have an effect on the residual bond capacity, in particular for relatively small concrete covers. This is related to the more prominent concrete cracking in case of higher heating rates and, consequently, a higher probability of concrete splitting in the residual state. For the same reason it appears to be unconservative to use the results obtained from slow-rate heating to directly evaluate the post-fire behavior of bond e.g. based on the estimated temperature level close to the rebar. Further research is required in order to quantify the effects of heating rate on the bond behavior.

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BOND OF GFRP BAR AND CONCRETE: NUMERICAL APPROACH

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ABSTRACT

Current practice of FRP (Fibre Reinforced Polymer) RC (Reinforced Concrete) structures mainly takes perfect bond as an assumption for modelling the structural behaviour. As distinct to that, present work deals with application of local bond-slip law for estimating the influence of the interaction of FRP bar and concrete at structural level. For this purpose, the investigation involves two scales. Small scale (pull-out test specimens) and full-scale (reinforced concrete beams) are considered for assessment of numerical models. 3D finite element (FE) model was developed to consider the non-linear behaviour of the interface, as well as of the concrete, using the commercial software Abaqus. The bond interface properties were simulated considering the secant modulus-based damage evolution law using cohesive elements. The developed bond model, validated by pull-out tests, was subsequently adopted for the numerical predictions of the mechanical response of full-scale beams. The model showed great potential to simulate the behaviour of structural element reinforced with such FRP materials, particularly at ultimate loading level.

1 Introduction

Since few decades ago, corrosion developed in RC structures was recognized as a huge detrimental factor that causes spalling of concrete, consequently affecting serviceability and ultimate resistance of such structural elements. It significantly decreased durability and safety of RC structures and increased the maintenance and reconstruction costs. Therefore, the idea of FRP rebars used as the reinforcement for concrete structures raised owing to their non-corrosive nature, as a key superior characteristic over the steel bars. However, different material properties resulted in many unknown aspects when dealing with this kind of reinforcement in concrete. Although already applied, especially in the construction of small-size bridges, there are still unknown issues that largely influence the use of FRP bars in wider range. Bond of FRP bar and concrete represents one of these issues, mainly due to many different materials and bar surfaces in use, as well as unknown behaviour under long-term and combined environmental effects, such as cyclic loading, alkaline solutions, or high temperature effects.

One simple but effective method for experimentally assessing the bond properties of reinforcement bar in concrete is the pull-out test, recommended by Rilem. Despite the simple setup, pull-out test is in wide use since it gives preliminary conclusions and serves as an input parameter when investigating the bond on macro level, as a part of overall structural performance.
FEM (Finite element method) has been widely used for simulating the real structural behaviour and consequently for giving the prediction about the response of the structure under different geometric, loading or boundary conditions. Although debonding is one of the typical failure modes in FRP reinforced concrete structures, in many research studies, it is neglected and structure is simulated using FEM assuming perfect bond\textsuperscript{4}. This approximation can lead to satisfactory result, depending on the structure layout such as concrete cover or reinforcement ratio\textsuperscript{5}. The lack made by using a perfect bond is usually overcome with tension stiffening effect, which takes into account post-cracking tensile strength of concrete\textsuperscript{6}. However, tension stiffening is the effect that decrease with loading, while bond-slip does not, which implies that both need to be included in the model for achieving consistent results\textsuperscript{7}. Therefore, accounting for more exact and precise results, especially when exploring the structural response under ultimate load, as well as performing parametric studies about the influence of different parameters on structural behaviour, needs the bond performance to be included into modelling and simulation of global structural response.

Present paper uses the numerical methodology assessed on pull-out test level for accounting the bond behaviour on the full-scale level. Namely, experimental results gained from pull-out tests were successively predicted by modelling the bar and concrete interface with cohesive elements. Subsequently, same methodology is used to simulate debonding behaviour inside a GFRP reinforced concrete beam, which was subjected to monotonic three-point bending loading up to failure. Perfect bond between bar and concrete is modelled for comparison purposes. The main asset of the paper was to explore the possibilities of use a local bond-slip model for simulating the structural response, when dealing with this new type of reinforcement. The outcomes of the analysis, comparisons and global conclusions are discussed onwards.

2 Damage evolution for the FRP bar-concrete bond

The present method includes the use of bond damage evolution approach for describing the FRP bar-concrete interface behaviour. It assumes that ascending branch of shear stress-slip curve represents undamaged state of behaviour and models it as linear. Linear behaviour is considered up to the peak point of the curve that designates maximum bond stress ($\tau_b$) and damage initiation point. This point is followed by propagation of damage, described by damage evolution law. Scalar damage variable $D$ is used for defining the damage evolution. The bond shear stresses ($\tau_s$) are affected by the damage variable as follows (Eq. 1):

$$
\tau_s = \begin{cases} 
\tau_e, & \tau_e \leq \tau_b \rightarrow D = 0, \text{no damage} \\
(1-D)\tau_e, & \tau_e > \tau_b 
\end{cases}
$$

where $\tau_e = K_{bc}\delta$

where $\tau_e$ is the bond shear stress predicted by elastic law, without damage, considering the initial elastic bond stiffness ($K_{bc}$) and the relevant slip ($\delta$).

Present paper uses secant modulus-based model for estimation of parameter $D$ and simulation of damage evolution. This model modifies the value of $D$ depending on the ratio between secant bond
stiffness \( (K_{\text{sec}}) \) and initial (elastic) bond stiffness \( (K_{\text{he}}) \), with respect to the corresponding slip (Eq. 2)

\[
D = \begin{cases} 
0, & \delta \leq \delta_b \\
1 - \left( \frac{K_{\text{sec}}}{K_{\text{he}}} \right), & \delta > \delta_b 
\end{cases}
\]

(2)

where \( \delta_b \) is the slip at the bond damage initiation.

Knowing the bond shear stress-slip curve from the experiment, values of \( D \) are obtained and damage evolution law is formulated.

3 Pull-out test

3.1 Experimental investigation

A comprehensive experimental campaign was performed with the aim to assess bond properties of GFRP (Glass Fibre Reinforced Polymer) bar and concrete. Eccentric pull-out test setup was adopted to measure experimentally the effect of two parameters: thickness of the concrete cover (10 and 20 mm) and concrete mechanical properties (cubic compression strength from 23 to 56 MPa). GFRP rebar (Schöck, ComBAR®) was made of unidirectional E-glass fibres and vinylester resin. It had nominal diameter of 8 mm and external ribbed surface cut into the bar after curing. Embedment length of the bar was selected as its 5 diameters.

Bar was pulled-out quasi statically and bar slip data was collected by transducers (LVDTs), while strain data on specimen’s front surface was collected by digital image correlation (DIC) technique. As a result, shear stress-slip curves were drawn, based on the assumption of uniform shear stress distribution along the embedment length. Moreover, DIC analysis revealed strain maps that were used for assessing the crack evolution in the concrete cover. Detailed explanation about the experimental investigation and results can be found in Ref.8.

3.2 Numerical modelling and results

The experimental pull-out specimens were numerically simulated using Abaqus software1. The bar/concrete interface was simulated by cohesive elements. Relevant bond properties were defined according to the aforementioned secant modulus-based model. Cohesive element had negligible thickness of 0.001 mm. Both main surfaces of cohesive element were tied to the surrounding surfaces of bar and concrete, using a surface-based tie constraint (no-slip occurrence allowed). More details about modelling procedure are available in Ref.9.

The predictive performance of the numerical model was validated by comparing to the shear stress-slip curves derived experimentally. Figure 1 presents the comparison of experimental and numerical results for a concrete of compressive strength 23.3 MPa and covers of 10 and 20 mm (S1C10 and S1C20, respectively). It is evident that the numerical FE simulation predicts accurately the experimentally derived results of pull-out tests. Moreover, the crack pattern obtained from the experimental and numerical results is compared in case of specimen S1C10, and it showed good
accordance in terms of crack propagation and damage concentration for different load levels (Figure 2). Presented results yield strong confidence in the numerical model and its predictive performance.

Figure 1: Comparison of experimental and numerical shear stress - slip curves.

Figure 2: Experimental (top) and numerical (bottom) maps of the maximum principal strain for different load levels

4 Three–point bending of a GFRP reinforced concrete beam

4.1 Experimental investigation

For the application of the interface model to a real scale structural element, a GFRP reinforced beam was considered, previously experimentally investigated\(^1\). The adopted GFRP bar had external surface wrapped with a spiral yarn and sanded with a quartz sand (Figure 3). The bars had nominal diameter of 16 mm, longitudinal tensile modulus of elasticity and strength of 39 GPa and 885 MPa, respectively. Concrete had cylindrical compression strength of 49.8 MPa and tensile strength of 4 MPa attained by indirect tensile test\(^1\).
Figure 3: GFRP bar Ø16, with sanded and spirally wrapped external surface.

Since another type of the bar was used, again the preliminary bond characteristics were experimentally assessed, with pull-out test.

The reinforced concrete beam had 150 cm supports span and it was loaded quasi statically in three-point bending configuration at room temperature. The reinforcement distribution and the experimental setup is detailed in Ref.\textsuperscript{11} and presented in Figure 4.

Figure 4: Scheme of the beam and experimental setup.

Beam was leaning against two metallic cylinders and the load was applied in the middle of the span, with a hydraulic jack, through 300x200 mm contact surface. The mid-span displacement of the beam was measured continuously along with loading with the two LVDTs placed on its top surface. Collapse occurred due to shear failure (Figure 5).

Figure 5: Failure mechanism of specimen.
4.2 Numerical modelling and results

Modelling of the beam in the present paper is based on 3D FEM. For the sake of comparison, perfect bond between bar and concrete and bond model of the interface were considered. Perfect bond is modelled by using tie connection that doesn’t allow slipping between bar and adjacent concrete surface during the loading. As distinct from this, for simulating the bar/concrete interface, cohesive elements were used as described for pull-out simulations (Section 3.2).

Constitutive behaviour of concrete was modelled using concrete damage plasticity (CDP) introduced by Lubliner et al. and implemented in Abaqus. CDP parameters are set as the software default due to the lack of the relevant experimental data. Compressive and tensile behaviour were modelled according to Model Code recommendation that defines stress-strain and stress-crack opening law, respectively. Maximum concrete strengths are taken as the reported values by experimental work. The CDP model allows visualization of damage assuming that cracking initiates at points where the tensile equivalent plastic strain is greater than zero, so this method will be used for representation of damage pattern.

FRP material is modelled as linear elastic transversally isotropic, with elastic constants derived from experimental data and the Chamis rules.

Bond law calibration is based on the experimental results of the corresponding pull-out test (Figure 6) that uses the bar and concrete with same properties as the ones in the beam experiment. Experimental data were fitted with polynomial function of second order to obtain the prediction of debonding behaviour over the larger slip. Although this pull-out test used significantly shorter embedment length comparing to effective length of the bar in the beam, this is neglected in the present simulations. The elastic bond stiffness is defined from initial slope of bond stress-slip curve and is 80 GPa. Maximum bond shear stress of 12.64 MPa is taken from pull-out test, as well as damage evolution law based on secant modulus method.

![](image)

Figure 6: Shear stress – free-end slip curve derived from pull-out test.

For the sake of decreasing the calculation time, the beam geometry is simplified by modelling only one quarter of it, using the property of two planes of symmetry (Figure 7). The loading and boundary
conditions were modelled similar to the ones adopted in the experimental program. Concrete and bar are modelled with 3D solid hexahedral elements C3D8R that use reduced integration, while cohesive elements are modelled using cohesive elements COH3D8. A refined mesh was applied in the zones where relatively high stress gradients are expected to develop.

As first output of the numerical analysis, the load versus mid-span displacement curves of the beam with perfect bond (PB) and the beam with cohesive elements (CE) are compared with the experimental curve (Figure 8). It can be seen that both numerical models provide similar trend to the experimental one. However, initial (pre-cracking) stiffness and concrete cracking strength of numerically modelled beams exceeds the corresponding ones in the experimental specimen. The reason is probably due to the possible small-extent cracking in the beam created during the shrinkage and curing. Specific points on the diagrams are designated with numbers 1 to 5 and explained together with damage evolution in the beam (Figure 9, Figure 10).
Figure 9: Numerical model with cohesive elements at interface: maps of tensile equivalent plastic strain.
Figure 9 and Figure 10 detail the damage development on the outer side of the model with CE and PB, respectively, for different load levels as detailed in Figure 8. Namely, point 1 and 2 (Figure 8) designate the local drop in bearing capacity of the beam, which can be attributed to the formation of two flexural cracks (Figure 9). Point 3 depicts tendency of the second flexural crack to transform partly into shear crack, which indicates the beginning of final failure process of the beam. This flexural-shear crack was propagated toward another flexural crack, as long as in point 4 CE beam attains its failure load, after which its bearing capabilities decrease, and finally in point 5 there is a drastic drop of bearing resistance. Active bond length of the bar was detected as the part where bar strain abruptly decreases to zero and here it is located in the shear span of the beam, up to the support. Active bond length does not change much after the point 3, i.e. initiation of shear cracks. Therefore, final failure mode of the beam was mainly due to the shear cracks.

Perfect bond beam follows the similar scheme as CE, until reaching the point 4 on the diagram. Despite connecting of flexural and shear cracks, the beam still yields its bearing capacity from additional force transfer to the bar. Consequently, PB beam reaches its ultimate capacity at significantly higher mid-span displacement level, point 5 (Figure 8).
5 Conclusions

Present paper deals with two FE model scales for numerical investigation of GFRP and concrete bond. Firstly, the pull-out test specimens were modelled by adopting the damage evolution method. Then, the same strategy of bond behaviour simulation was subsequently applied to a full-scale beam. For comparison purposes, perfect bond model (no bond-slip occurrence) was adopted as well. Outcomes of the experimental and numerical investigations were compared and following conclusions are derived:

- Damage evolution law according to secant modulus-based model for estimation of bond damage parameter was shown successful for modelling of bar/concrete debonding behaviour,
- Calibrating the bond damage model using the corresponding experimental bond tests, in combination with the proper material constitutive models, yielded good results in pull-out tests simulation in a sense of accurate bond-slip and crack pattern prediction,
- Three-point loading of GFRP reinforced beam was successfully simulated as well, yielding more accurate results, comparing to perfect-bond beam model. The advantage of bond-slip model is visible at ultimate (failure) load phase. Simulating the bond-slip performance leads to accurate prediction of beam deflection corresponding to failure and failure load as well.

The current paper confirmed that modelling the bond interface effect plays an important role in the structural behaviour of FRP reinforced concrete. Therefore, bond effect showed as significant factor to be accounted for designing and simulating FRP RC structural response.

6 Acknowledgement

The research was developed in the framework of the Marie Curie Initial Training Networks – “endure” European Network for Durable Reinforcement and Rehabilitation Solutions, project no: 607851. Schöck Bauteile GmbH is gratefully acknowledged for supplying the GFRP rebars.

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RETROFITTING & STRENGTHENING
ANCHORAGE INELASTICITY AS AN IMPORTANT FACTOR AFFECTING THE RESPONSE OF REINFORCED CONCRETE FRAMES WITH STEEL BRACING

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ABSTRACT

Installation of a diagonal steel brace, in a reinforced concrete (RC) framed structure, is one of the popular ways of lateral strengthening of the structure. The method is often used in seismic retrofitting and strengthening of structures. The bracing is designed assuming a rigid connection with the RC frame. In order to transfer the forces between the bracing and the RC frame members, post installed anchorages are normally designed.

During earthquakes, high demand in terms of load and crack cycling are imposed on anchors that may push them in inelastic range. Thus for a realistic evaluation of performance of a braced frame, it is required to consider the inelastic behavior of anchorages as well. In the present work, a simple, yet realistic spring model for inelastic behavior of anchorages from literature (INFASO Project1) is used for development of a modelling approach for non-linear analyses of RC braced frames. The modelling approach can be compatibly employed in commercial finite element software’s like SAP 2000, which are typically used in civil engineering design. The modelling approach is validated against available experiment in literature. The study clearly highlights the importance of consideration of inelasticity of anchorages in realistic performance evaluation of RC frames braced with steel bracing.

1 Introduction

With the increasing technical know-how and the consequent updating of the guidelines for design and construction of reinforced concrete (RC) structures, there is always a requirement to strengthen and retrofit the existing RC structures. This requirement is particularly more critical for non-seismically detailed structures which find themselves in regions where new requirements of seismic demands are imposed in the process of updating the guidelines. Several options2 are available for strengthening and retrofit of an existing RC structure, with special consideration given to enhance the lateral load carrying capacity of the structure. In cases where it is required to increase the strength and stiffness of the RC framed structures under the action of lateral forces, the engineer has an option to add a masonry wall, or an RC shear wall or a brace. The underlying principle in each of these techniques is to change the load transfer mechanism from moment resisting frame to a predominant truss mechanism.
Installation of diagonal steel braces in RC frames is a practical and quick method of retrofitting and strengthening. It is because of the practical ease that this method has gained popularity. The design of the retrofit solution in this case, however, involves a challenge for design of the anchorage connection between the steel brace and the existing RC frame. The connection is assumed to be rigid in civil engineering design, and is expected to transfer high amounts of tensile and shear forces, while maintaining this rigidity. As an alternate to avoid this criticality in the local connection, the recommended suggestion is installation of steel columns through the floor diaphragms. This method however, is more invasive and relatively complex.

In case the engineer decides to design a concentric diagonal steel brace connected directly to the RC frame through gusset plates using post installed anchorages, it is essential to ensure the desirable performance through non-linear analyses. It is customary to assume anchorages as rigid in such analyses. In performance analyses of steel braced RC frames reported in literature, non-linear behavior of the RC frame members (beams, columns) and of the steel brace is considered, while assuming that the anchorage behaves in a rigid manner. This paper focusses on the importance of consideration of anchorage inelastic behavior in the performance analysis of a steel braced RC frame. Because of the limited information available on this subject, the development of a rationalized modelling approach for considering the effect of anchorages at progressively different levels of complexity is presented. The approach recommended in the present work is validated against available experiment in the literature.

2 Benchmark Experiment for Validation

The data from experiments reported by Mahrenholtz, et.al. is used for validation of the modelling approach which is developed in the present work. The experiments involved structural testing of full scale RC frame with and without bracing under pseudo static cyclic loading. A sketch of the set up is shown in Figure 1. Details of the geometric section of the structural members and the related material properties are provided in the test reports. In order to connect the bracing to the RC frame, an arrangement of gusset plates anchored to the RC frame using bonded expansion anchors (M24) with embedment depth of 225mm was considered. A group of four anchors each was used to connect the gusset plate to the beam and column section.

A comparison of the load-displacement relationship of the bare frame with the braced frame observed experimentally clearly highlighted the superior behavior of the braced frame. The braced frame was reported to exhibit a failure in the anchorages used to connect the bracing to the RC frame. An assumption of rigid anchorages cannot simulate the realistic performance of such frames which have been designed using the state-of-the-art design practices. This paper adopts a progressive modelling approach for consideration of anchorage behavior in the increasing order of complexity while comparing the numerical solution with the results from the benchmark experiment. In this process, a rationalized modelling approach including inelastic modelling of anchorages is developed for realistic simulation of the performance of a RC frame braced with a diagonal steel bracing. It should be noted that the present work focusses on the modelling of anchorages. Hence, the direction of loading on the RC frame is considered for the case when the bracing is in tension. The other direction of loading with bracing in compression (which leads to buckling effects in the bracing and
direct bearing of the gusset plate on the RC frame members) has not been considered in the present work.

![Figure 1: Test set up for the experiments](image)

3 Modelling Approach and Validation

The progressive modelling for considering inelasticity of the RC-frame-brace-anchorage system in the increasing order of complexity is presented in this section. The results from the numerical analysis of the model at each modelling stage are compared with the experiment, and the deficiency of each state of modelling is highlighted.

3.1 Bare frame modelling

For modeling of inelastic behavior of RC frame members on a global structural level, a lumped plasticity approach is usually followed\textsuperscript{7,8}. The inelasticity is modelled through hinge characteristics at critical locations which are susceptible to maximum demand forces and moments. The procedures for arriving at these localized hinge characteristics are well documented in the literature\textsuperscript{7,8}. Typically the definition of hinge characteristic consists of a load/moment capacity versus deformation/curvature/rotation relationship. For beam section the hinge characteristics are generally defined by moment-rotation relationship. For columns, an interaction of axial forces with inelastic moment-rotation characteristics is typically defined. The applicability of these procedures has been validated against numerous experiments\textsuperscript{8}. A bare frame model with the localized plastic hinges for the beam and column sections was prepared in SAP 2000 and subjected to non-linear static (pushover) analyses.

The pushover curve (base shear versus roof displacement) compared with the envelope of the cyclic curve of the bare frame from the experiment is shown in Figure 2. The failure mode indicated a ductile failure by localization of damage in the beam followed by final failure in the columns as observed in the experiment. It is thus concluded that the state-of-the-art practices for modelling of inelasticity of RC sections as available in literature\textsuperscript{7,8} can be relied up on for a realistic simulation of the overall behavior of a bare RC frame.
3.2 Present state approach: Braced frame-direct connection

In the present state-of-the-art practices, the anchorages are assumed to be rigid. Hence in the structural modelling anchorages are completely ignored, and the brace is connected to the intersection node of the beam and the column directly (referred to as direct connection in the present work). In this case the non-linear behavior of the brace is modeled using a localized plastic hinge governing the axial force-displacement relation obtained with procedures available in literature\textsuperscript{7,8}.

Figure 2: Comparison of load-displacement curve: experiment (solid) with pushover curve (dotted)

Figure 3: Progression of failure mode and comparison of load displacement curve for direct connection of brace to the RC frame
The progression of failure modes and the comparison of the pushover curve with the envelope of the cyclic curve of the braced frame (when the bracing is in tension) from the experiment is shown in Figure 3. It is seen that the model with direct connection exhibits higher initial stiffness when compared to the observation in the experiment. This is attributed to the fact that the flexibility on part of the anchorages is not considered at all. The failure mode initiates with yielding of the brace element followed by formation of hinges in beams and the columns. The post peak behavior is also not correctly simulated using this approach. Thus it is seen that the present state approach of direct connection can provide reasonable estimates of forces for braced frames. However, with regards to the displacement behavior and the overall performance and progression of failure modes, the model provides incorrect information.

3.3 Modelling anchorage location

It was attempted to account for the behavior of anchorage in a progressive way in the present work. At first it was essential to consider the location of anchorages. For this purpose each of the two planar groups of anchorages was modelled by a single element connecting the rigid gusset plate to the centerlines of the beam and the column (Figure 4a). The localized inelastic hinges for the beam and column sections on either side of the anchorage connection were defined. The inelastic behavior of the brace element was also considered as in the earlier case.

![Figure 4: Modeling of location of anchorages and corresponding load-displacement comparison](image)

The line element used to define each of the two groups of the coplanar anchors was assigned equivalent elastic characteristics for tension and shear. The elasticity of anchorage group under tension and shear load was calculated based on the equivalent steel section of the anchorages. The modelling of location of elastic anchorages introduced additional rigidity in the model which reflected as a higher failure load and stiffer load-displacement behavior as compared to the direct connection. The load-displacement curve for this case is shown in Figure 4b. The failure mode was more similar to the direct connection case.

3.4 Anchorage Inelasticity Approaches

As a next step in the progressive modelling approach, the inelasticity of anchorages was also accounted for in the model. Modelling of inelasticity of anchorages, requires information on the overall inelastic load displacement behavior. Although several force based models\(^9\)\(^{-11}\) for evaluation of inelastic behavior of anchorages are available in literature, there is a general scarcity of the information of the related displacement behavior. A simplified spring modeling for considering the
In this work, the force based approach in EN-1992-4, and as prescribed by Berger’s Model, and the force-displacement approach from INFASO have been considered. The displacement relation corresponding to the ultimate inelastic force capacities as prescribed by Sharma were used for the two force based approaches. A comparison of the anchorage force displacement relation for the coplanar group by the three approaches is shown in Figure 5. The main difference in the force based approaches in EN-1992-4, and as prescribed by Berger’s Model is in the way the beneficial effect of stirrups is considered in the inelastic modelling for anchorages when concrete cone failure is the governing failure mode.

The EN-1992-4 approach conservatively considers the effect of stirrup failure through yielding or bond action. The resistance component applicable to the anchor by the stirrup alone is considered, while the resistance offered by concrete is neglected. As a consequence, the ultimate capacity of the anchorage group for the anchorage failure mode evaluated using this approach is a gross underestimation (as seen in Figure 5). A comparison of load displacement behavior of the braced frame model with anchorage inelasticity considered using EN-1992-4 approach is shown in Figure 6a. The failure mode indicated the failure of anchorages leading consequently to a bare frame action.
The approach for inelastic modelling of anchorage in EN-1991-4 is thus a conservative one. This approach, although suited for design purposes, does not serve the purpose of realistic capacity evaluation and should not be used in performance analysis.

The ultimate capacity model for anchorages prescribed by Berger\(^9\) accounts for the resistance from the concrete as well as from the stirrups. The proposal can be expected to provide more realistic estimates of the ultimate inelastic capacity of the coplanar anchorage group. A comparison of load-displacement curves with anchorage inelasticity considered as per the proposal from Berger\(^9\) is shown in Figure 6b. The numerical analysis is seen to capture the loading curve reasonably well till the peak. However, the post peak behavior is not correctly captured. This can be attributed to the redistribution of forces after the peak, which is strongly affected by the displacements corresponding to the different load levels of the anchorages.

As a next step of study, the inelastic load displacement model for anchorages evaluated using the procedure detailed in the INFASO report\(^1\) was used to model the anchorage inelasticity. A comparison of the force-displacement curve from numerical analyses employing the INFASO model\(^1\) for inelasticity of anchorages and the experiment is shown in Figure 6c. It is seen that the overall load displacement behavior is captured in a reasonable manner using this approach of modelling the anchorages. The progression of failure mode obtained in the analysis also compared well with that observed in the experiment.

### 4 Conclusion

The purpose of this paper was to highlight the importance of modelling the inelastic behavior of anchorages for a realistic structural performance evaluation of steel braced RC frames. Although the concept of direct connection of steel element to the RC frame nodes provides reasonable estimates in terms of forces, the approach is found to be incorrect in the estimation of displacement behavior as well as simulation of failure modes. This is because the redistribution of forces after the peak load cannot be realistically simulated by such numerical models because of non-consideration of anchorage behavior.

A simplified procedure for consideration of anchorage inelasticity has been briefly explained in this paper. Through a progressive modelling approach, the best available method for consideration of the inelastic behavior of anchorages was arrived at. The inelastic load-displacement model prescribed for anchorages by INFASO project was found to be well suited for anchorage modelling in performance evaluation of the steel braced RC frame considered in the present work. The model is simple enough to be implemented in commercially available analyses and design software. The use of this proposed modelling approach will lead to a more realistic performance evaluation of steel braced RC frame structures connected with post installed anchorages.

### References:


ABSTRACT

The present paper deals with strengthening of existing reinforced concrete structures in shear by means of either post-installed reinforcing bars (“rebars”) or a specifically designed anchor system. The avoidance of restrictions for the use of a structure during installation, the efficiency of the applied strengthening method, economic aspects and simplicity of installation were main targets in this research.

Post-installed rebars are commonly used when, during the erection process, newer parts of an emerging structure are cast against already existing sections and loads have to be transferred via the joints. Above that the completion, extension and retrofitting of existing structures as well as strengthening of members in the sense of adding more reinforcement are of main interest. The applicability of post-installed rebars for strengthening slabs against punching shear failure [1] or beams and slabs for increasing their shear resistance [2] has been proven in a series of experimental campaigns. Post-installed rebars are thereby inserted into pre-drilled and mortar-injected holes from the bottom side of the concrete member and anchored with metal plates at the accessible bar end. Depending on the number of rebars, the load enhancement factor varied between 1.3 and 1.6.

Recently the prototype of a new anchorage system was designed especially for this type of application and tested in large scale shear tests with 5.4 m long precast T-beams [3]. The anchor is set vertically into the pre-drilled hole and the load is then introduced at the very end inside the structure by means of a self-undercutting expansion sleeve. With the application of the post-installed anchors a 40% load increase with four and nearly a 90% increase with seven anchor rods was observed.

1 Introduction

There exist only few well established methods to strengthen RC structures in shear so far. Compared to flexural strengthening, retrofitting of shear deficient members is rather challenging for the following main reasons:

- The strengthening system shall be reliably activated without the necessity of large additional live loads or deformations of the structure.
Currently there are no harmonized recommendations for RC shear design itself available, therefore the rather complex design of shear strengthening is even more disputable.

Local shear strengthening can typically be done by applying externally bonded reinforcement or steel plates, near surface mounted reinforcement or externally anchored or grouted steel bars inserted into holes drilled through the cross section. Nowadays Fibre Reinforced Plastic (FRP) materials are increasingly used for externally bonded or near surface mounted reinforcement due to their extraordinary strength to weight ratio, high durability and ease of application.

However, the mentioned current shear strengthening methods have some major disadvantages such as significant intervention in the existing structure, high installation efforts or restrictions on the use of the structure during construction works. The methods presented in here can overcome these drawbacks and their effectiveness has been verified in a number of tests on RC beams subject to shear loading. Since both post-installed rebars and undercut anchors are set from the soffit of the RC member, traffic disturbances can be avoided to a large extent. Both presented strengthening systems have been tested over the last 10 years in several series of tests performed either in the accredited laboratory of the Hilti Corporation or in the certified building lab of the Carinthia University of Applied Sciences in Austria, hereafter being summarized. As both techniques have been tested independently in different research projects, the test setups differ. More detailed information on the projects is given in the literature referred to below.

2 Post-installed rebars for strengthening existing members

Post-installed reinforcement is reinforcement that is installed into hardened concrete members by drilling holes and installing the bars with adhesive mortars. For strengthening and rehabilitation projects, as well as in specific situations in new construction, post-installed bars are used more and more frequently.

In the case of strengthening existing structures, they may serve as an additional subsequently applied reinforcement. The effectiveness of post installed reinforcement for strengthening in shear and also punching shear was verified in a number of structural tests [1, 2]. The post installed reinforcement was hereby always installed from the bottom side of the concrete members and inserted into pre-drilled and mortar-injected holes. Neighbouring parts of a structure are thereby not damaged and work on the decking zones is avoided so that the usability during construction works is not significantly affected.

2.1 Experimental program

The overall testing program and the varying parameters are listed in Table 1 (more details are given in [2]). The test specimens consisted of RC-beams as outlined in Fig. 2 with a shear slenderness \( a/d = 3.4 \). The effectiveness of such kind of bonded reinforcement depends strongly on the available anchorage length which requires an inclined installation (preferably 45° as chosen here).

The number and location of the inclined post-installed rebars ø16 (yield strength \( f_{y} \approx 560 \text{ MPa} \)) has been varied as well as the type of injection mortar for bonding-in the reinforcement. The rebars in this case have been anchored with metal plates at the accessible bar end on the bottom side; the top
end of the rebar was glued in at the upper side of the beams. Two mortars approved for anchorages of post-installed rebar were used, one cementitious anorganic and an organic epoxy-type mortar.

The first and the last test were performed as reference tests without subsequent strengthening. The mean concrete cube strength was about 45 MPa in all tests. The tests were performed upside-down, i.e. the load introduced in upwards direction. As reference for the maximum achievable load with such kind of elements in one test (test no. 8) anchor plates without slip on top and bottom side were provided.

Table 1: Setup and results of tests with post-installed reinforcement

<table>
<thead>
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<th>beam</th>
<th>bars w strain gauge</th>
<th>bars</th>
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<th>s₂ [cm]</th>
<th>s₃ [cm]</th>
<th>Resin type***</th>
<th>Fₘₐₓ</th>
<th>Fₘₐₓ,i / Fₚₑᵣ ₁₁₀</th>
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<td>-</td>
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<td>237</td>
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<td>25,6</td>
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<td>ep b</td>
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</table>

*anchor plates on top and bottom side. **inner reinforcement bar

***cem = cementitious, ep b = epoxy based

Figure 1: Setup of tests with post-installed reinforcement [2]

2.2 Test results

All tested beams failed due to a premature shear rupture. The beam bending resistance was not decisive and the yield strength of the upper tensile reinforcement never reached.

Both injection mortars used for bonding-in the post-installed rebars proved to be suitable, however, the epoxy resin leading to a better utilization of the rebar tensile capacity: Based on the evaluation of the strain measurements, in the case of the epoxy resin the inner bars 1 and 2 reached more than 80% of their yield strength on average whereas the bars bonded-in with the cementitious mortar achieved
roughly 60% of the yield strength. In terms of overall shear resistance, with the epoxy resin an average load increase of 56% and with the cementitious mortar an increase of 46% has been recorded.

Some additional observations concerning development and propagation of cracks can be summarized as follows:

- In each test the first cracks developed in the flexural tension zone perpendicular to the edge and quickly progressed to the inner web zone of the beam. The crack inclination changed from 30 to 45° in the web zone to a very smooth angle when the crack finally approached the compression flange on the bottom side.

- With increasing load, in the zone of the upper longitudinal tension reinforcement the main cracks more and more took course parallel to the reinforcement bar, activating the bar’s dowel action resistance.

- Even with shear crack widths larger than 1 mm still a significant load increase was observed.

- At failure typically one main crack with an average inclination of ~30-40° opened progressively, the crack width finally reaching about 2-3 mm (Fig. 2).

![Figure 2: Shear failure mode and full beam deflection curve at different load levels (test 8)](image)

3 **Strengthening with sleeved undercut anchors**

In a more recent project funded by the Austrian Research Promotion Agency (FFG) several methods of effective shear strengthening have been investigated. Among other techniques, together with the Hilti Corporation a new anchorage system was designed especially for this type of application: An undercut anchor is set vertically into a pre-drilled hole and the load is then introduced locally by means of a self-undercutting expansion sleeve (see principle in Fig. 3). This working principle allows for a concentrated load introduction at the end of the anchor even in cases where the shear crack passes quite close to the end in an unfavorable way. In addition, a tubular sleeve around the whole length of the inner threaded rod, providing a non-bonded free length for the rod, enables pre-stressing of the anchor rod with a torque wrench.
3.1 Test setup
An adequate setup and shape of the cross section had to be chosen to guarantee typical shear failure even after application of the shear strengthening. The test specimen was a single span, 5.38 m long concrete beam arranged in a 3 point load setup (details see Fig. 3). Taking into account the distance of 1.585 m between load introduction and support, the shear span-to-depth ratio a/d was 3.24 (with an effective depth d of 489 mm).

In order to overcome premature flexural failure even in the case of enhanced shear load bearing capacity, an over-reinforced T-shaped cross section was designed, with four 30 mm and two 25 mm diameter bars as longitudinal tensile reinforcement situated at the bottom of the beam. The steel was a high grade S670/800 type, in order to ensure maximum flexural resistance with a still reasonable reinforcement degree. As top reinforcement, only four 14 mm diameter bars and two 10 mm diameter bars were foreseen in order to create a stable reinforcement cage.

No stirrup reinforcement was provided in the beam sections between load introduction and closest support where the subsequent strengthening was applied. Along the remaining length of the beam 12 mm diameter stirrups at 190 mm spacing were foreseen to enforce shear failure at the critical shear span.

![Figure 3: Test setup and beam cross section with applied undercut anchors (units: [mm])](image)

3.2 Material properties and anchor installation
The beam specimens were made of normal strength concrete C30/37. The mean concrete strength measured on 150 mm cubes on the day of testing was 43.5 N/mm².

The longitudinal reinforcing bars provided at the bottom of the beams had a nominal yield strength of 670 N/mm² with an average value 10% above. All other types of used reinforcement, i.e. longitudinal top bars and stirrups, were made of standard steel B550 B.

After a preloading phase with 10 load cycles at SLS-level, all specimens except for the reference test were provided with the strengthening elements. An overview of the tested constellations is given in Table 2. All strengthening elements were installed at a constant spacing of either 190 or 380 mm.

The undercut anchors were installed in 540 mm deep pre-drilled holes of 22 mm diameter which had before been injected with a high strength epoxy-type mortar (filling only the gap between sleeve and borehole). In the last phase of the setting process, the anchors were driven into the concrete by means...
of an appropriate hammer drill to produce the undercut. Afterwards they were immediately prestressed by applying a defined torque moment of 60 Nm. As the curing time of the resin in this case at 20°C room temperature was only 45 minutes, the test could usually be performed already on the same day.

The steel rod was provided with a metric screw thread size M 12. While this inner steel rod had a free length to activate a prestressing force, the tubular sleeve was bonded in the present tests by the epoxy-based injection mortar, thus providing an additional tensile capacity. Assuming full bond and load introduction of the ribbed tubular sleeve to the surrounding concrete, the tensile strength of one single undercut anchor yields then altogether about 108 kN (contribution of prestressed rod plus full tensile capacity of the tubular sleeve). However, concerning the sleeve contribution it should be mentioned that the full activation of the steel sleeve is only possible if the bond length is sufficient, which means that the critical shear crack does not cross the anchorage close to its end.

3.3 Results of shear tests with undercut anchors

All strengthened beam specimens exhibited a significant load increase due to the applied strengthening elements. An overview of the recorded loads and the load increase with respect to the reference test is given in Table 2.

The first single and slightly inclined shear cracks with an inclination of roughly 60°-70° to the horizontal axis appeared at the reference beam at a load level of about 130 to 140 kN, and at the strengthened beams such cracks showed up consistently at a significantly higher load level around 200 to 250 kN in between the strengthening elements.

At a load level of 300 to 350 kN, i.e. close to the failure load of the unreinforced reference beam (see load deflection curves below), the inclination of the propagating shear cracks started to decline to an angle between 30° and 45° to the horizontal axis. At the reference beam this was followed quickly by the formation of a continuous critical shear crack and, having passed through the flanged section, soon led to final failure. At the same load level the strengthened beams showed no significant loss in stiffness as the strengthening elements were more and more activated: pronounced shear cracks appeared increasingly in between the elements and at that stage the formation of a typical truss mechanism was initiating.
The load transfer via the truss mechanism was more and more identifiable and clearly developed above a load level of about 400 to 450 kN. Before total failure of the beams an increasing number of less inclined cracks appeared (around 25° to the horizontal axis), partly unifying and crossing the strengthening elements while showing an increasing crack width. Finally one or two main cracks started to propagate through the whole cross section.

Failure took place in such a way that a first inner rod broke and in most cases fell out of the sleeve. This behavior reveals a lack of ductility of the used prototype rods. Improved rods should allow plastic redistribution of loads once the first rod has reached its yield strength. However, this first rupture of one single rod never led to a total loss: after a sudden limited drop in the load, again some recovery of the load was observed until another rod broke. Final failure was usually reached with the rupture of the second or even another third rod. The bonded sleeves remained intact in the structure, thereby preventing a sudden total breakthrough of the concrete beam.

Table 2. Setup and results of shear tests with undercut anchors.

<table>
<thead>
<tr>
<th>Test</th>
<th>No. of elements</th>
<th>Spacing [mm]</th>
<th>$F_{\text{max}}$ [kN]</th>
<th>$V_{\text{max}}$ [kN]</th>
<th>$F_{\text{max}} / F_{\text{max,ref}}$</th>
</tr>
</thead>
<tbody>
<tr>
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</table>
Despite of the rather small cross section of each rod, impressive loads were achieved with the strengthened beams. The shear capacity was nearly doubled (enhancement factor 1.88) with the application of seven post-installed anchors, while a number of four rods still led to an average increasing factor of 1.46. Obviously the increase of the number of anchors from 4 to 7, corresponding to a reduction of the spacing from 380 to 190 mm, did not lead to a proportional increase of the shear load bearing capacity.

4 Comparison of the two strengthening methods

As mentioned before, the two strengthening systems, i.e. post-installed reinforcement and subsequently applied undercut anchors, were investigated in different research projects not linked to each other and at different times. Even though for that reason the test setups are not comparable, some general comparisons and observations can be summarized as follows:

- The efficiency of the strengthening systems can first of all be judged by the achieved load enhancement when comparing strengthened beams and reference beam: with the post-installed reinforcement bonded in with an epoxy resin, the load enhancement was between 144% (2 bars on each side) and 157% (average increase with 3 bars on each side). In the case of the undercut anchors the increase was between 146% (average increase with 4 bars) and 188% (average increase with 7 bars).

- As the absolute values of the achieved load enhancements are not directly comparable due to different setups, it is more meaningful to judge the efficiency on the basis of a very simple model used also by other researchers (see e.g. [4]): The maximum tensile strength of all strengthening elements crossed by the critical shear crack (in vertical direction) is summed up and related to the ultimate shear load in the test: \( k = \frac{\sum A_{ni,v} f_{um}}{V_{ult}} \). Comparing the tests where 3 post-installed reinforcing bars are applied with the tests with 4 undercut anchors, an efficiency factor \( k \) of 86% in the first and 81% in the second case yields. The factor of more than 80% is more or less confirmed in the first case by the strain measurements performed along the post-installed reinforcement (with the undercut anchors the geometry of the anchor did not allow for gluing strain gages). So in both cases the efficiency is comparable and with more than 80% can be rated as very high. Nevertheless note that this consideration is only a very rough approach as neither all geometric aspects nor additional effects such as dowel action of longitudinal reinforcement or rotational displacement of the rigid body are taken into account.

- Concerning the installation procedure, the bonded reinforcement requires inclined setting in order to increase the available bond length in the anchorage zone. The undercut anchors, on the other hand, can be set in vertical direction due to the concentrated load introduction at the anchor’s head.

- Even though in the present tests an injection mortar was used, the undercut anchors would also work without bond between sleeve and surface of borehole (however, the contribution of the sleeve would in that case be cancelled). In addition some pre-stressing of the inner rod is possible which allows for immediate activation and is helpful also for serviceability limit state considerations (e.g. crack control).
5 Conclusions and outlook

Strengthening in shear by means of bonded reinforcement or sleeved undercut anchors is an effective way to achieve significant load enhancements without the requirement of large interventions in the existing structure. As such kind of anchorage can be set from the bottom side of an RC member, the traffic flow is not affected by the construction works.

In the different shear tests performed with both strengthening techniques, in general a satisfying load increase between 30% and 90% (depending on number of anchors and setup) was achieved.

A direct comparison of both systems shows that the efficiency is quite similar. Nevertheless the undercut anchors demonstrate some advantages with respect to the setting process: While the bonded reinforcement has to be installed inclined in order to achieve sufficient anchorage length and cannot be loaded before the curing time of the injection mortar has passed, the undercut anchors succeed with their concentrated load introduction which enables immediate loading and in addition some pre-stressing effect when applying the torque moment.

Further investigations are required to judge the strengthening effect on shear reinforced structures as interaction with existing stirrup reinforcement will lead to some reduction of the total sum of the single load bearing capacities.

References:


PUNCHING
PUNCHING STRENGTH OF RC FOOTINGS WITH SHEAR REINFORCEMENT – EVALUATION OF CODE PROVISIONS AND COMPARISON WITH TEST RESULTS

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ABSTRACT

The punching shear design provisions according to current codes have either a semi-empirical (e.g. Eurocode 2 and Eurocode 2 along with National Annex of Germany) or a physical (e.g. Model Code 2010) background. Although the design procedures are very similar, the predicted punching shear capacities may differ significantly, depending on the background of the design provisions and the considered influences. Recent test series on reinforced concrete footings without and with high amounts of shear reinforcement are available and can be used for the evaluation and improvement of the current provisions. Nevertheless, the evaluation of the code equations for the design of the shear reinforcement (failure inside the shear-reinforced zone) is still not possible, since systematic test series with varying amount of shear reinforcement have not yet been conducted.

To investigate the punching shear behavior of reinforced concrete footings with low and medium amounts of shear reinforcement, a systematic test series consisting of five specimens with stirrups as shear reinforcement was conducted. In the tests, the amount of shear reinforcement was varied by changing the stirrup diameter. In this paper, the results of the test series are summarized and compared to the predictions according to Eurocode 2, Eurocode 2 along with the German Annex, and Model Code 2010.

1 Introduction

The punching shear behavior of reinforced concrete footings without and with high amounts of shear reinforcement has been investigated by various researchers in the past1-12. Considering these test results, the current design provisions according to Eurocode 213, Eurocode 2 along with National Annex of Germany14,15 and Model Code 201016 were established. Nevertheless, the evaluation of the code equations for the design of the shear reinforcement has not yet been possible, since systematic test series on reinforced concrete footings with varying amount of shear reinforcement are not available.
In the present study, a series of five punching tests on reinforced concrete footings without and with varying amount of shear reinforcement was conducted. The aim of the experimental program was the investigation of the transition from punching failure without shear reinforcement, to punching failure inside the shear-reinforced zone, up to failure on the level of maximum punching shear capacity. In this paper, the formation of inner shear cracks as well as the activation of the shear reinforcement is discussed depending on the amount of shear reinforcement. To evaluate current code provisions, the failure loads from the tests are compared with the predicted punching shear capacities according to Eurocode 2, Eurocode 2 along with National Annex of Germany and Model Code 2010.

2 Experimental program

2.1 General

The experimental program included five tests on reinforced concrete footings with conventional closed stirrups as punching shear reinforcement. The test parameter exclusively investigated in the test series was the amount of shear reinforcement. The variation of the amount of shear reinforcement in the tests was achieved by changing the stirrup diameter $\phi_w$. The notations DF28N (without shear reinforcement, test results already published by Siburg and Hegger), B3-6 ($\phi_w = 6$ mm), B3-8 ($\phi_w = 8$ mm), B3-10 ($\phi_w = 10$ mm) and DF35 ($\phi_w = 14$ mm, test results already published by Siburg and Hegger) were used for the test specimens.

2.2 Materials

For all test specimens, commercial ready mixed concrete was used. The maximum coarse aggregate size was 16 mm. The concrete mixture was designed to reach a cylinder strength of $f_{c,\text{cyl}} = 20$ MPa at the day of testing. To prevent premature failure, the column stubs consisted of high-strength concrete with concrete compressive strengths between $f_{c,\text{cyl}} = 90$ and 106 MPa. Additionally, the column stubs were strengthened with a steel collar made of 10 mm steel plates. For all specimens, German steel B 500B was used as reinforcing steel for the flexural reinforcement and for the shear reinforcement. Table 1 summarizes all test parameters, dimensions, failure loads and material properties.

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<th>$d$ [m]</th>
<th>$c$ [m]</th>
<th>$b$ [m]</th>
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<th>$\phi$ [mm]</th>
<th>$f_y$ [MPa]</th>
<th>$\rho_l$ [%]</th>
<th>$A_{sw}$ [cm$^2$]</th>
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<tr>
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<td>18.9</td>
<td>558</td>
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<tr>
<td>DF35</td>
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<td>19.4</td>
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<td>0.86</td>
<td>111</td>
<td>14</td>
<td>0.62</td>
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<td>12649</td>
<td>5850</td>
<td></td>
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</tbody>
</table>

$h$: slab thickness; $d$: effective depth; $c$: square column dimension; $b$: square footing dimension; $a_l/d$: shear span-depth ratio; $f_{c,\text{cyl}}$: concrete compressive strength; $\phi$: diameter of flexural reinforcement; $f_y$: yield strength of flexural reinforcement; $\rho_l$: flexural reinforcement ratio; $A_{sw}$: cross-sectional area of shear reinforcement up to 0.8$d$; $\rho_w$: shear reinforcement ratio according to LIPS; $f_{yw}$: yield strength of shear reinforcement; $V_{\text{flex}}$: shear force that produces flexural failure according to yield-line theory; $V_{\text{test}}$: ultimate failure load.
2.3 Test specimens
The test series consisted of five reinforced concrete footings with identical dimensions. The square side dimension was \( b = 2700 \) mm and the slab thickness was \( h = 650 \) mm. The square column stubs had side dimensions of \( c = 300 \) mm and were casted monolithically at the center of the footings. The distance from the extreme concrete compression fiber to the centroid of tension reinforcement (effective depth) varied between \( d = 580 \) and 594 mm, resulting in a shear span-depth ratio of approximately \( \frac{a_{\lambda}}{d} = 2.0 \) (with \( a_{\lambda} \) being the distance from the column face to the edge of the footing). The column perimeter-depth ratio was \( \frac{u_{0}}{d} = 2.0 \). The flexural reinforcement ratio was set to \( \rho_{f} \approx 0.86 \% \) to prevent flexural failure especially for the specimens with high amounts of shear reinforcement.

The layout of shear reinforcement of specimens B3-6, B3-8, B3-10, and DF35 is shown in Fig. 1. The first and second row of shear reinforcement consisted of 44 stirrups and the third and fourth row consisted of 28 stirrups. In accordance with German Annex of Eurocode 2, the distance between the first perimeter of shear reinforcement and the column face was set to \( s_{0} = 180 \) mm \((0.3d)\). The radial spacing between the other perimeters of shear reinforcement was \( s_{r} = 300 \) mm \((0.5d)\).

![Figure 1: Geometry and layout of flexural and shear reinforcement](image)

2.4 Test setup and measurements
The test specimens were loaded by a uniform surface load using the test setup described in SIBURG AND HEGGER\(^9\) and KUERES ET AL.\(^{19,20}\). The footings were tested upside down. The uniform soil pressure was simulated with 25 load application points. Twelve hydraulic jacks transferred their load through cross beams to two load points each. A further hydraulic jack with a piston area of half the size completed the load arrangement above the column. All hydraulic jacks were linked to a common manifold and applied the same load independent of the displacement. In order to avoid any formation of membrane forces in the test specimens, polytetrafluorethylene (PTFE)-coated sliding and deformation bearings of dimensions 140 x 140 mm were placed between the footing and the beams.

During testing, several measurements were performed to investigate the punching shear behavior of the specimens. The vertical displacement of the test specimens was recorded at the corners of the
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column stub, the footing’s corners, and several other locations using linear variable differential transformers (LVDTs). To investigate the development of the inner shear cracks, the increase in slab thickness was measured at several points and the penetration of the column into the slab was monitored. Also, the slab rotation was measured at the edges of the specimens at up to four locations. Strain gages were used to measure the strains in the flexural reinforcement at up to eight locations and in the shear reinforcement at several points. To obtain the average strain at the bar’s center of gravity, two strain gages were attached to opposite side faces of the reinforcing bars at each measuring point. The concrete strains were recorded at up to six locations on the compression face of the footing near the column.

2.5 Test procedure
The load was applied under load control in increments of 250 kN. To simulate lifetime loading, the load was cycled ten times between a calculated service load \( V_{\text{Service}} \) and half its value. The service load varied between 1900 kN (DF28N) and 2800 kN (DF35). After the load cycles, the test specimens were continuously loaded until failure took place.

3 Experimental results

3.1 General
All tests failed in punching of the footing. Before failure occurred, increasing slab thickness, increasing strains in the shear reinforcement and penetration of the column stub into the slab were observed. The failure loads \( V_{\text{test}} \) are summarized in Table 1. The comparison with the flexural capacities of the footings \( V_{\text{flex}} \) according to the yield-line theory\(^{18}\), as well as the recorded flexural reinforcements strains clearly below the yield strain, prove that the flexural capacities of the specimens were not reached when failure occurred due to punching.

3.2 Cracking characteristics
After testing, the footings were sawn into four quarters to examine the inner shear crack formation (Fig. 2). Depending on the amount of shear reinforcement, the observed crack patterns differ significantly. In specimen DF28N without shear reinforcement (\( \rho_w = 0\% \)) only one shear crack formed, which had an inclination of approximately 35° (Fig. 2(a)). A shear crack with a similar inclination was observed in specimen B3-6 with a very low amount of shear reinforcement (\( \rho_w = 0.11\% \), Fig. 2(b)). Due to the spacing of the stirrups, the first and second row of shear reinforcement were crossed by the shear crack and activated, which can be confirmed by the performed strain measurements. Compared to specimen B3-6, in specimen B3-8 with twice as much shear reinforcement (\( \rho_w = 0.20\% \), Fig. 2(c)) a further, more steeply inclined shear crack crossing the first row of shear reinforcement developed. While in specimen B3-8 both critical shear cracks had a similar crack width, the width of the more steeply inclined crack was much higher in specimen B3-10 (\( \rho_w = 0.31\% \), Fig. 2(d)). Also, the saw-cut of specimen B3-10 shows more finely distributed cracks in the region of the column. The saw-cut of specimen DF35 with the highest amount of shear reinforcement (\( \rho_w = 0.62\% \)) revealed a crack pattern, which is typical for a failure on the level of maximum punching shear capacity. As can be seen in Fig. 2(e), the failure crack had a very steep inclination and propagated between the first row of shear reinforcement and the column face.
3.3 Strains in punching shear reinforcement

Fig. 3 shows exemplarily the measured tensile strains in the shear reinforcement in the first two rows. The strains corresponding to the yield strength of the shear reinforcement are indicated in the diagrams. In the tests, substantial steel strains were first observed at a load level coinciding with the beginning of the inner shear crack formation. This was confirmed by the measured increase in slab thickness at approximately the same load level. In specimen B3-6 ($\rho_w = 0.11\%$), the first two rows of shear reinforcement were crossed by the shear crack (Fig. 2(b)) and reached the yield strength prior to failure (Fig. 3(a,b)). The fact that the load-increase after the activation of the shear reinforcement was relatively low indicates that the amount of shear reinforcement was only slightly higher than required to resist the forces at crack formation state (minimum reinforcement). Compared to specimen B3-6, in specimen B3-8 ($\rho_w = 0.20\%$) the load-increase caused by the shear reinforcement was more pronounced, even though the shear reinforcement in the first and second row exceeded the yield strength (Fig. 3(c,d)). A similar activation of the shear reinforcement in the first row was observed for specimen B3-10 ($\rho_w = 0.31\%$, Fig. 3(e)). The activation of the second row of shear...
reinforcement was less pronounced (Fig. 3(f)). In contrast, the strains in the shear reinforcement of specimen DF35 (\(\rho_w = 0.62\%\)) did at no point exceed the yield strength prior to failure (Fig. 3(g,h)).

Figure 3: Measured strains of shear reinforcement in first and second row
3.4 Changes in thickness of the specimens

The changes in thickness of the specimens were continuously measured during the tests at several positions. The results at failure are shown in Fig. 4. Regardless of the amount of shear reinforcement, the highest changes in thickness were measured close to the column face. While specimen DF28N without shear reinforcement failed in a brittle manner without large measured changes in thickness (Fig. 4(a)), a more ductile behavior at failure and higher changes in thickness were observed in the specimens with shear reinforcement (Fig. 4(b-e)). In this context, specimens B3-6 ($\rho_w = 0.11\%$, Fig. 4(b)), B3-8 ($\rho_w = 0.20\%$, Fig. 4(c)), and B3-10 ($\rho_w = 0.31\%$, Fig. 4(d)) showed considerable changes in thickness up to the third row of shear reinforcement. The highest changes in thickness were observed in specimen B3-10. In contrast, the changes in thickness in specimen DF35 ($\rho_w = 0.62\%$, Fig. 4(e)) were much smaller compared to B3-10 and concentrated between the first row of shear reinforcement and the column face.

Figure 4: Measured changes in thickness of the specimens at failure
3.5 Failure mode

The punching failure modes of the specimens can be determined combining the evaluation of the crack patterns (Fig. 2), the tensile strains in the shear reinforcement (Fig. 3), and the changes in slab thickness (Fig. 4). In this context, the saw-cuts indicate that an initial, relatively flat inclined shear crack developed in each specimen. While in specimens DF28N ($\rho_w = 0\%$) and B3-6 ($\rho_w = 0.11\%$), the initial shear crack directly triggered the punching failure (without shear reinforcement or inside the shear-reinforced zone), the amount of shear reinforcement in specimen B3-8 ($\rho_w = 0.20\%$) was sufficient to redistribute the forces at crack formation state leading to an additional, more steeply inclined shear crack. Nevertheless, strain measurements in the shear reinforcement in the first two rows as well as the measured changes in slab thickness prove a punching failure inside the shear-reinforced zone for specimen B3-8. In contrast, in specimen DF35 ($\rho_w = 0.62\%$) a very steep failure crack developed between the first row of shear reinforcement and the column face. The fact that the first row of shear reinforcement did not reach the yield strength and that the second row of shear reinforcement was hardly activated, indicates that the capacity of the shear reinforcement was not reached when failure occurred on the level of the maximum punching shear capacity. According to the crack patterns and the steel strains in the second row of shear reinforcement (no yielding), the punching failure mode of specimen B3-10 ($\rho_w = 0.31\%$) may be situated in the transition between a punching failure inside the shear-reinforced zone and a failure on the level of the maximum punching shear capacity. Nevertheless, the fact that the first row of shear reinforcement yielded prior to punching indicates a punching failure inside the shear-reinforced zone, since the capacity of the shear reinforcement was reached when failure occurred.

4 Comparison of predictions and experimental results

The main parameter investigated in the present test series was the amount of shear reinforcement. Fig. 5 depicts the normalized failure loads $V_{\text{test}} / (u_{0.5d}d^2(\rho_l f_{\text{ck}}))^{1/3}$ (where $u_{0.5d}$ is the control perimeter in a distance 0.5$d$ from the column, $d$ is the effective depth, $\rho$ is the flexural reinforcement ratio, and $f_{\text{ck}}$ is the characteristic value of the concrete compressive strength) for specimens DF28N ($\rho_w = 0\%$), B3-6 ($\rho_w = 0.11\%$), B3-8 ($\rho_w = 0.20\%$), B3-10 ($\rho_w = 0.31\%$), and DF35 ($\rho_w = 0.62\%$) depending on the shear reinforcement ratio $\rho_w$. With increasing amount of shear reinforcement, the normalized failure loads increase. While the load-increase up to a shear reinforcement ratio of $\rho_w = 0.20\%$ (specimens DF28N, B3-6, and B3-8) is more than linear, the load-increase is less pronounced for ratios $\rho_w > 0.20\%$ (B3-10 and DF35) and seems to result in an upper limit (s-form of capacity curve).

For the sake of comparing the test results with current punching shear design provisions, Fig. 5 shows the calculated punching shear capacities of a similar footing with varying shear reinforcement ratio according to Eurocode 2 (EC2), German Annex of Eurocode 2 (EC2NA), and Model Code 2010 (MC2010). For the calculations, all material and strength reduction factors in the design equations were taken as unity.
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Figure 5: Comparison of predictions and experimental results

The development of the predicted normalized punching shear capacity with increasing shear reinforcement ratio according to the provisions of EC2, EC2NA, and MC2010 is shown in Fig. 5. As indicated by the predictions based on EC2NA (solid grey line) and MC2010 (dashed black line), starting at the lowest capacity (without shear reinforcement), the predicted punching shear capacity increases with increasing amount of shear reinforcement until a certain maximum punching shear capacity is reached. A further increase of the shear reinforcement ratio does not lead to an increased punching shear capacity according to the codes. While the predicted punching shear capacity without shear reinforcement and the maximum punching shear capacity are comparable according to EC2NA and MC2010, the transition between these cases differs significantly. EC2NA requires the shear reinforcement in the first two rows to carry the whole shear force (no concrete contribution). In contrast, MC2010 assumes a combination of concrete and steel contribution depending on the footing’s rotation. In the present case, the punching shear capacity according to EC2 is governed by the capacity of the compression struts at the column face due to the low concrete compressive strength and the small column. Hence, increasing the amount of shear reinforcement does not lead to an increased punching shear capacity according to EC2. Further information regarding the various punching shear provisions as well as comparisons and evaluations are available in literature21-23.

The punching shear capacity without shear reinforcement (specimen DF28N) and on the level of the maximum punching shear capacity (specimen DF35) are predicted reasonably well by the provisions according to EC2NA and MC2010. Nevertheless, the transition between these capacities is defined in a rather conservative manner. While MC2010 requires in the present case a shear reinforcement ratio of approximately $\rho_w = 1.10\%$ to reach the maximum punching shear capacity, specimen DF35 already reached this capacity with a shear reinforcement ratio of $\rho_w = 0.62\%$. The punching shear capacity according to EC2NA is not affected by a shear reinforcement ratio up to $\rho_w = 0.40\%$, but rapidly increases until the maximum punching shear capacity at $\rho_w = 0.60\%$ is reached. The conservatism of the code provisions is especially evident for specimens B3-6, B3-8, and B3-10, since the installed amounts of shear reinforcement do not or only slightly increase the predicted punching shear capacity according to the various codes.

The comparison of predictions and experimental results indicates, that the punching shear provisions according to EC2, EC2NA, and MC2010 considerably underestimate the punching shear capacity of
footings with shear reinforcement. In this context, the limitation of the maximum punching shear capacity to the capacity of the compression struts at the column face according to EC2 seems not to reflect the actual punching behavior of these members. This was already observed in previous databank evaluations\textsuperscript{23}. Also, the assumption of EC2NA to neglect a concrete contribution for the design of the shear reinforcement seems to be rather conservative. MC2010 assumes that the punching shear capacity of reinforced concrete slabs is governed by the slab rotation\textsuperscript{24}. The application of the provisions on shear-reinforced concrete footings indicates that the slab rotation might not be the decisive parameter for these members, since vertical deformations allow for a good activation of the shear reinforcement also for small rotations.

5 Conclusion

The results of the experimental investigations on reinforced concrete footings with varying amount of shear reinforcement and the comparison with code provisions allow the following conclusions to be drawn:

- The present test series allows for the investigation of the transition between punching failures without shear reinforcement and failures on the level of the maximum punching shear capacity. While specimen DF28N failed in punching without shear reinforcement and specimen DF35 failed on the level of the maximum punching shear capacity, specimens B3-6, B3-8, and B3-10 failed inside the shear-reinforced zone.

- The inner shear crack formation of reinforced concrete footings is strongly influenced by the amount of shear reinforcement. While the punching failure of the specimens without or with very low amounts of shear reinforcement was directly triggered by the initial, relatively flat inclined shear crack, the specimens with higher amounts of shear reinforcement were capable of redistributing the forces at crack formation state, which led to additional, more steeply inclined shear cracks. For the specimen with very high amounts of shear reinforcement, a very steep failure crack developed between the first row of shear reinforcement and the column face.

- The activation of the shear reinforcement depends on the inner shear crack formation. In this context, in the tests with low amounts of shear reinforcement the first two rows were crossed by the failure crack and substantially activated (yielding of shear reinforcement). With increasing amount of shear reinforcement, the measured strains in the second row of shear reinforcement decreased due to the formation of steeper inclined shear cracks.

- With increasing amount of shear reinforcement, the failure loads increase. While the load-increase up to a certain shear reinforcement ratio is more than linear, the load-increase is less pronounced for larger ratios and seems to result in an upper limit (s-form of capacity curve).

- The punching shear provisions according to Eurocode 2, German Annex of Eurocode 2, and Model Code 2010 significantly underestimate the punching shear capacity with low and medium amounts of shear reinforcement, which leads to a safe but uneconomic design. Further research (e.g. more compact footings with \(a_i/d < 2.0\)) is required to investigate if this is generally true.
6 Acknowledgement

The presented work was part of a research project from the German Concrete and Construction Technology Association (DBV) at the Institute of Structural Concrete of RWTH Aachen University (IMB). The research project was funded within the program for sponsorship by the German Federation of Industrial Research Associations (AiF, IGF number 18114 N/1) of the German Federal Ministry for Economic Affairs and Energy on the basis of a decision by the German Bundestag.

References:


PUNCHING SHEAR STRENGTH OF PLATE DOWELS IN CONCRETE INDUSTRIAL GROUND FLOORS

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ABSTRACT

Steel dowels are used in adjacent concrete slabs on-ground to minimise their relative vertical displacement. Steel plate dowels are an alternative to steel bar dowels and are becoming more popular in use for industrial ground floors. A failure mode of the steel plate dowels is punching shear failure which can be characterised by the development of cracks from the sides of plate dowel to the surface of concrete slab. One of the parameters affecting the design of plate dowels is the horizontal gap between the slabs which will reduce the strength of plate dowel. This reduced capacity is influenced by the reduction of the concrete fracture surface and the eccentricity of applied load. In this paper, results of an experimental program that investigates the effect of eccentricity of load on punching shear strength of steel plate dowels are presented. Further, a non-linear finite element software for concrete structures, ATENA, is used to predict the failure mode. The effects of parameters such as concrete compressive and tensile strength on the punching shear strength are explored using ATENA. Finally, the guidelines of Technical Report 34¹ (TR34) for punching shear strength of steel plate dowels are compared to the results from experiment and finite element analysis. Eighteen physical tests were completed for this comparison.

1 Introduction

Round and square dowel bars have been the common practice for load transfer and control of relative vertical displacement between adjacent concrete slabs on-ground. They are commonly used in street, road, highway and airport pavements. However, the requirements of joint performance for dowels of industrial floors are different to the pavement slabs due to the sizes of industrial floors. It is common to have industrial floors extend more than 30m in each direction as compared to 8m width of road pavement slabs². The use of plate dowels with plastic sleeves not only controls the relative vertical displacement of adjacent slabs but also allows for horizontal movements of the slab parallel to the joints. Industrial floors have an increased need to minimise vertical displacement between slabs due to higher transport speeds of vehicles travelling across them carrying heavy loads. The plate dowels and plastic sleeves are commercially available in different shapes such as rectangular, circular or diamond shapes. Figure 1 shows a plate dowel and its plastic sleeve. The sleeve would be installed to the concrete formwork before the cast of the first slab and after removal of the formwork, the dowels
are placed in the sleeves before the casting of adjacent slab. In most cases the plate dowels are
installed in the mid-thickness of the slabs.

The unrestrained movement of the slabs in both horizontal directions, as shown Figure 2, would
minimise the cracking of slabs due to concrete shrinkage.

![Figure 1 A plate dowel with the plastic sleeve](image)

![Figure 2 Unrestrained movement of slabs in horizontal directions](image)
To design the load transfer capacity of dowel plates, engineers use the guidelines of TR34\(^1\). The design criteria cover the steel plate capacity under shear and flexure actions. Further, it provides guidelines to check the concrete capacity in shear and bearing strength. Often the critical load capacity is governed by the punching shear strength of the concrete which is the subject of this paper.

2 Background

TR 34\(^1\) follows Eurocode 2\(^3\) (EC2) to calculate the punching shear strength of dowels. The critical shear perimeter is located at distance \(2d_{\text{eff}}\) from the edges of the dowel, where \(d_{\text{eff}}\) is the effective depth equal to 0.75\(d\) as shown in Figure 3. In most cases, no reinforcement is provided to increase the punching shear strength of the slabs and the minimum shear strength on the critical perimeter is calculated by equation (1).

\[
v_{Rd,c} = 0.035 \cdot k_s \cdot f_{ck}^{0.5}
\]  

(1)

Where: 
\(k_s = 1 + (200 / d_{\text{eff}})^{0.5}\) ≤ 2 and 
\(f_{ck}\) is the characteristic cylinder concrete compressive strength.

And, the predicted punching shear strength of plate dowel, \(N_{u,\text{pred}}\) is calculated by equation (2).

\[
N_{u,\text{pred}} = v_{Rd,c} \cdot u_{cr} \cdot d_{\text{eff}}
\]  

(2)

Where: \(u_{cr}\) is the critical perimeter as shown in Figure 3.

The effective depth \(d_{\text{eff}}\) in the third edition of TR 34\(^4\) is the distance between the plate and the surface of slab, \(d\), as shown in Figure 3. In the fourth edition of TR 34\(^1\), the effective depth reduced to 0.75\(d\). The 25\% reduction of effective depth is recommended to compensate for the eccentricity of the load.

![Figure 3 Shear critical perimeter around the dowel at distance 2d_{eff}](image)
3 Experimental results

The scope of the experimental programme was to investigate the effect that eccentricity of applied load has on the punching shear strength of the plate dowel. Joint gaps of 6mm, 10mm and 20mm were considered for both plate dowels with a plastic sleeve and without a plastic sleeve. A sample size of three was used for each test, where 18 tests were carried out in total. Concrete compression cylinder test specimens were tested between the period of 28 to 32 days after cast, where the average concrete cylinder compressive strength was measured as 38.5MPa. The thickness of the concrete slabs was 150mm and dowels were placed centrally in the thickness of slab. A reinforcement mesh of 8mm diameter bars with 200mm spacing was placed in the bottom half of the test slabs. A schematic view of the test setup is shown in Figure 4. The 110mm x 110mm plate dowel thickness 10mm was used for the tests. Steel yield strength of the plate dowel material was 325MPa.

The mode of failure for all the tested specimens was concrete fracture in the shape of a half cone as shown in Figure 5. The cracks started from the sides of the dowel with average angle of 33° for \(a_1\) and \(a_2\), shown in Figure 5a, and cracks at the rear of the cone exhibited average angle of 68°, \(a_3\) as shown in Figure 5b.

The observed ultimate load at failure, \(N_{u,test}\), is compared to the predicted failure load calculated using the fourth edition of TR 34\(^1\), \(N_{u,pred}\), and presented against different joint gaps as shown in Figure 6. The predicted failure loads are safely below the observed ultimate failure loads for different joint gaps.

![Figure 4 Test setup for punching shear strength of plate dowels](image)

![Figure 5 a) Front view of concrete fractured surface, b) Side view of concrete fractured surface](image)
The Average, Standard Deviation (SD), Coefficient of Variation (CoV) and lower characteristic value for the ratio of $N_{u,\text{test}}/N_{u,\text{pred}}$ from the fourth edition of TR34$^1$ and the third edition of TR34$^4$ are presented in Table 1. The lower characteristic value corresponds to the 5percent fractile of the mean ultimate loads for a confidence level of 90percent assuming a normal distribution. Equation (3) from AS3850$^5$ is used to calculate the lower characteristic value $X_k$.

$$X_k = \bar{x} \cdot (1 - k \cdot \text{CoV}) \quad (3)$$

Where $\bar{x}$ is the mean value and $k$ is the sampling factor. For the sample size of eighteen in a normal distribution, $k_s$ is 2.25.

![Figure 6 Ratio of the observed ultimate failure load to the predicted failure load by TR 34$^1$](image)

### Table 1 Statistical coefficients, lower characteristic and lowest value for $N_{u,\text{test}}/N_{u,\text{pred}}$

<table>
<thead>
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<th>$N_{u,\text{test}}/N_{u,\text{pred}}$</th>
<th>Average</th>
<th>SD</th>
<th>CoV (%)</th>
<th>Sampling factor</th>
<th>Lower characteristic</th>
<th>Lowest value</th>
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<td>1.16</td>
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<td>8.1</td>
<td>2.25</td>
<td>0.59</td>
<td>0.62</td>
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Table 1 shows the 25percents reduction of the effective depth in TR34$^1$, fourth edition, as compared to TR34$^4$, third edition, results in a safer prediction of failure load.
Although the experimental results exhibit higher failure load than the predicted values by the fourth edition of TR34, the characteristic value of 0.95 indicates the need for increasing the number of experimental tests, assuming the CoV is maintained or improved through additional tests.

4 Finite Element Modelling

Finite Element (FE) analysis is used to investigate the performance of plate dowel connections using ATENA, a nonlinear FE programme for concrete material analysis. The test setup, shown in Figure 4, was modelled in ATENA for a 10mm horizontal joint gap and the concrete compressive cylinder strength of 38.5MPa. The plate dowel was displaced 0.05mm vertically, shown in Figure 7, at each FE loading step applied through an infinitely stiff plate until the ultimate concrete failure load was reached. The load and displacement was monitored at the applied load face of the stiff plate during the analysis.

The concrete is categorised as grade C30 in Model Code 2010. For concrete grade C30, the lower characteristic, upper characteristic and mean tensile concrete strength are $f_{ctk,\text{min}}=2\text{MPa}$, $f_{ctk,\text{max}}=3.8\text{MPa}$ and $f_{ct,\text{mean}}=2.93\text{MPa}$ respectively according to Model Code 2010. In Figure 8, the ratio of the load from FE analysis over the average ultimate load from the testing, $N_{FE}/N_{\text{um,test}}$, is plotted against the monitored displacement for various concrete tensile strength of C30.

The lower characteristic tensile strength, $f_{ctk,\text{min}}=2\text{MPa}$, shows the best match to the average ultimate load from experiment.
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Further, various concrete grades, C25 to C45, with their corresponding lower characteristic tensile strength were used in the FE model. The ultimate predicted punching shear load for the plate dowel in concrete grades C25 to C45 were calculated according to the fourth edition of TR34\textsuperscript{1}, $N_{u,pred}$ and FE analysis, $N_{u,FE}$. In Table 1 Table 2, the ratio of $N_{u,FE}/N_{u,pred}$ is given for concrete grades C25 to C40.

<table>
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<th>Concrete grade</th>
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<tr>
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<tr>
<td>C30</td>
<td>1.27</td>
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<td>1.34</td>
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<td>C45</td>
<td>1.36</td>
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</table>

Table 2 shows that the fourth edition of TR34\textsuperscript{1} conservatively predicts the ultimate load for various concrete grades. The predicted failure loads by TR34\textsuperscript{1} become more conservative as the concrete grade is increased.
5 Conclusion and recommendations

The experimental programme shows the reduction of effective depth for calculating punching shear from $d$ to $0.75d$, shown in Figure 3, is suitably conservative.

For the tested plate dowel, failure mode for joint gaps 6mm, 10mm and 20mm in a 150mm thick concrete slab displays the characteristics of a punching shear failure.

Although all the experimental results showed a higher punching failure load as compared to the predicted values by the fourth edition of TR34\textsuperscript{1}, further experimental testing is recommended to increase the lower characteristic strength to a value greater than the predicted strength. A recommendation is to increase the sample size and hence lowering the sampling factor in equation (3).

FE analysis using ATENA suitably predicts the punching shear strength of the plate dowel with increased accuracy when the lower characteristic of concrete tensile strength is used.

The results from FE analysis using ATENA shows the predicted values by the fourth edition of TR34\textsuperscript{1} gives a safe ultimate load prediction for concrete grades C25 to C45. The predicted failure load becomes more conservative with the increase of concrete grade.

6 Acknowledgement

Ramsetreid, ITW Australia, for their continued generous support of our experimental work.

Cervenka Consulting, Prague, Czech Republic, for their assistance and extensive knowledge on non-linear numerical simulation of concrete structures.

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SLAB-COLUMN CONNECTION WITH EFFECTIVE LATTICE SHEAR REINFORCEMENT

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ABSTRACT

Reinforced concrete flat slabs supported by columns may fail by punching shear in form of a cone-shaped concrete breakout. To avoid this failure and to increase the shear resistance of a flat slab, shear reinforcement in the slab adjacent to the support can be used. A special punching shear lattice reinforcement with stiff anchorage and optimised inclination of the load bearing bars is able to increase the shear resistance of a slab to more than double compared with a slab without any shear reinforcement. This Filigran®-Punching Shear Reinforcement system is especially used in semi-precast flat slabs made of precast elements about 50 mm thin with in situ concrete cover. In this application gaps between the precast part and the column edge might be present. Previous experimental investigation showed no influence from a variation of the gap width on the shear resistance when this lattice shear reinforcement is placed close to the column. In addition to this detail, the upper surface of a precast column might end on a level above the lower surface of the slab. Such initial penetration of the column into the slab might reduce the height of the punching failure cone and thus the punching shear resistance of a slab. A full scale punching shear test on a 260 mm thick slab was carried out with a column penetrating 20 mm. The comparison with a test without column penetration showed no reduction of the punching shear strength. An additional comparison of these two tests with two other tests with semi-precast slabs reinforced with double headed studs confirmed the robustness of the column-slab connection reinforced with this Filigran®-Punching Shear Reinforcement which has to be placed close to the column.

1 Semi-Precast Flat Slabs

Semi-precast slabs are made of thin precast plates (fig. 1) with a thickness of about 50 mm and an in situ concrete layer on top. The prefabricated plate is reinforced with lattice girders, which ensure sufficient stiffness during transportation and erection on site. In the final stage the lower chord of the lattice girder functions as part of the bending reinforcement and the struts of the lattice girder bear as shear reinforcement.

In the early years semi-precast slabs were mainly used in one-way spanning slabs. Meanwhile semi-precast slabs are also provided as two-way spanning slabs. In this case the longitudinal reinforcement of one direction is put into the precast plates and the bending reinforcement perpendicular to this direction is laid down on the precast elements on site.
The application of flat slabs is limited due to its punching shear capacity. The punching shear resistance of a semi-precast slab could be influenced by the interface between the precast and the in-situ concrete layer. Moreover, the joints between several precast elements as well as between the elements and the column raise the question of impact on the load bearing behaviour. Therefore, punching shear tests to determine the maximum shear resistance of shear reinforcement should take into account these effects.

First full scale tests\(^1\) with lattice girders as punching shear reinforcement in composite slabs were carried out with a type of lattice girder, which has been used as shear reinforcement in one-way span slabs for many years. These lattice girders were arranged parallel to each other with the ascending lattice struts towards the column. The upper flexural reinforcement was placed on top of the upper chord. According to these tests, an increase of the punching shear resistance, depending on the concrete class, of 20% to 30% was approved in Germany.

Eligehausen et al.\(^2\) tested special lattice girders with stiff anchorage of the loadbearing bars and with loops protruding over the upper chord. These loops protruded into the layer of the upper flexural reinforcement. The punching shear resistance of composite slabs was increased by these girders by a factor of between 1.79 and 1.83\(^3\). The approved increase factor based on the German design standard was 1.7.

2 The Filigran®-Punching Shear Reinforcement

2.1 Full Scale Punching Shear Tests

Furche et al.\(^4\) introduced a highly effective lattice punching shear reinforcement. This Filigran®-Punching Shear Reinforcement FDB II system (fig. 2) has two longitudinal lower chords and one upper chord with a diameter of 10 mm. The strut diameter is 9 mm. The characteristic yield strength of the steel is 500 MPa. The characteristic (5%-quantile) ratio of the tensile strength over yield strength is 1.05 and the elongation at maximum strength is 2.5%. All bars are ribbed.
Five full scale tests\textsuperscript{4} on inner column slab connections were carried out to determine the maximum shear resistance. The slab thickness varied between $h = 180$ mm and 360 mm and the associated effective depths between $d = 145$ mm and 295 mm. Columns with square cross-sections ranged from $c = 240$ mm to 300 mm. This resulted in column perimeter to effective depth ratios of $u_0/d = 4.0$ to 8.0. The ratios of the upper bending reinforcement were chosen between $\rho_l = 0.70\%$ and $1.47\%$. The shear reinforcement elements were placed parallel to each other. In these tests a sufficient amount of shear reinforcement was chosen to avoid steel failure. Only in this case it is possible to determine the maximum strength of the compressive concrete strut, which characterises the maximum shear resistance according to several design models.

The test specimens were produced as composite slabs in two horizontal layers, with butt joints between the precast slabs and different distances between the precast elements to the column. The distance between the precast plates to the prefabricated column varied between 20 mm and zero. In one case the precast plates were placed 10 mm onto the column. Figure 3 shows the arrangement of the precast parts before concreting the top concrete layer. The edge of the precast slab was situated directly adjacent to the column. This arrangement was assumed to be crucial with respect to the load bearing capacity.
As a result of the tests the permissible arrangement of the Filigran®-Punching Shear Reinforcement and the precast parts of the slab is specified in a European Technical Approval\(^5\) according to figure 4. The required arrangement of the reinforcement close to the column edge in conjunction with the slight 20 mm inclination of the strut nearest to the column ensures that even steep punching shear cracks are penetrated by this strut.

![Permissible arrangement of the Filigran®-Punching Shear Reinforcement and precast slabs adjacent to the column\(^5\)](image)

**Figure 4**: Permissible arrangement of the Filigran®-Punching Shear Reinforcement and precast slabs adjacent to the column\(^5\)

### 2.2 Maximum Shear Resistance According to Eurocode 2

According to the design concept of Eurocode 2, the maximum punching shear strength is given as a multiple of the shear resistance of a reinforced concrete slab without punching shear reinforcement. According to the German adaptation of this standard\(^6\), a control perimeter around the column at a distance of 2\(d\) is used to prove the shear resistance (fig. 5a). This design concept is applied in the European Technical Approval ETA-13/0521\(^5\) for the Filigran®-Punching Shear Reinforcement FDB II as well as for double headed studs\(^7\). For this control perimeter the design resistance of concrete slabs without shear reinforcement is given by equation (1). The shear resistance depends on a factor \(k\) for size effect, the ratio \(\rho\) of flexural reinforcement and the characteristic compressive concrete strength \(f_{ck}\). This equation applies for inner columns, slabs without normal stress and column perimeters greater than 4\(d\) and less than 12\(d\). Equation (1) is here simplified without showing the minimum punching shear strength. Therefore, and for other boundary conditions, please refer to the mentioned standard\(^6\) or the ETA-13/0521\(^5\) respectively.

\[
v_{Rd,C} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho \cdot f_{ck})^{1/3} \tag{1}
\]

- \(v_{Rd,C}\) = design punching shear resistance (MPa)
- \(C_{Rd,c} = 0.18 / \gamma_c\) (partial safety factor for concrete \(\gamma_c = 1.5\))
- \(f_{ck}\) = characteristic concrete strength
- \(k = 1 + \sqrt{(200/d)} \leq 2\)
- \(d\) = effective depth (mm)
- \(\rho\) = ratio of longitudinal reinforcement
  - \(\rho = \sqrt{(\rho_z \cdot \rho_y)}\)
  - \(\leq 0.02\)
  - \(\leq 0.5 f_{cd}/f_{yd}\)

\[
v_{Rd,max} = \alpha_{max} \cdot v_{Rd,C} \tag{2}
\]
The maximum punching shear resistance for slabs with punching shear reinforcement is given by equation (2). The increase factor $\alpha_{\text{max}}$ in this equation depends on the type of shear reinforcement. Full scale tests with the Filigran®-Punching Shear Reinforcement gave ratios of the tested resistance to the expected resistance according to equation (1), taking into account a characteristic concrete strength of $f_{\text{ck}} = f_{\text{cm}} - 4$ MPa with $f_{\text{cm}}$ taken as the average strength of the tested cylinders. The individual values were between $\alpha_{\text{max},i} = 2.16$ and $\alpha_{\text{max},i} = 2.42$. A characteristic value of $\alpha_{\text{max}} = 2.09$ was evaluated as the 5%-quantile with a probability of 75%. Thus equation (3) is approved in ETA-13/0521.

$$v_{\text{Rd,max}} = 2.09 \cdot v_{\text{Rd,C}}$$

(3)

### 2.3 Maximum Shear Resistance According to ACI 318-14

According to the design concept of ACI, the punching shear strength must be proved in a critical section at a distance of 0.5$d$ from the column. An inner column with square cross section without openings or free edges nearby leads to a critical section as shown in figure 5b. For critical sections for other boundary conditions, please refer to ACI 318. The stress corresponding to two-way shear strength $v_c$ provided by normal concrete is given by equation (4). In contrast to equation (1), there is no flexural reinforcement ratio or size effect taken into account.

$$v_c = \frac{1}{3} \sqrt{f'_{\text{c}}},$$

(4)

Furche et al. evaluated the full scale tests with Filigran®-Punching Shear Reinforcement FDB II and compared it to the expected figures according to equation (4). The specified compressive strength of concrete $f'_{\text{c}}$ as the average value $f_{\text{cm}}$ of the tested cylinders was taken into account. The individual ratios of tested resistance to expected resistance achieved between $v_{\text{test}}/v_c = 2.15$ and 2.55 with a characteristic value of $v_{\text{test}}/v_c = 2.12$. Therefore, the maximum two-way shear resistance $v_u$ of slabs with Filigran®-Punching Shear Reinforcement can be taken as that of double headed studs according to equation (5) which gives double the shear capacity of slabs without shear reinforcement.

$$v_u = \frac{2}{3} \sqrt{f'_{\text{c}}}.$$
It was shown in tests that the shear resistance increases with the flexural reinforcement ratio. An evaluation of tests with a ratio of greater than 1% only gives a higher characteristic increase factor of 2.27. If flexural reinforcement of this amount is provided, a higher maximum two-way shear resistance of $v_u = 0.75 \sqrt{f_{c'}}$ can be endorsed.

3 Column Precast too High

3.1 The Problem

Columns are typically concreted prior to pouring the slab. Depending on the execution quality, the upper surface of the precast column may be situated on the level of the lower surface of the slab or deviating from it. Independent of the execution method (monolithic or composite slab), a column cast too high penetrates into the subsequently poured slab. According to some design recommendations, the effective height $d$ of the slab should be reduced to $d_v$ (fig. 6) in such cases. According to this approach, the critical shear crack ends at the upper edge of the column. This assumption is based on cases of failures of monolithic concrete slabs without shear reinforcement where an initial penetration of the column was found subsequently. The author has no knowledge of any experimental investigations to study this effect on the punching shear capacity of the slab. It might well be the case that the assumption according figure 6 is conservative, especially in cases of slabs with punching shear reinforcement. On the other hand, it could happen that in composite slabs a column concreted too high leads to a small gap between the precast slab and the column. Such a gap could also have a negative effect on the punching shear capacity.

![Figure 6: Assumption of reduced effective depth in case of too high concreted column to be proven](image)

3.2 Punching Shear Tests

In punching shear tests with composite slabs it was already shown that placement of precast slabs directly near the column has no negative effect on the loadbearing capacity. An additional test was carried out to investigate the behaviour in case of columns concreted too high. In addition to this, the question was raised whether composite slabs reinforced with double headed studs show a similar behaviour.

Furche described a test with the Filigran®-Punching Shear Reinforcement in a composite slab with an overall thickness auf 260 mm. The distance between the precast part and the column was 20 mm and the upper surface of the column was situated on the level of the lower surface of the slab (fig. 7a, left side). This test was taken as a reference test. In a new test, the upper column surface was situated 20 mm above the lower surface of the slab and the precast part was placed flush with the column (fig. 7a, right side). The Filigran®-Punching Shear Reinforcement in both arrangements was situated
close to the edge of the precast slab part. This ensured the maximum distance of the loadbearing bar to the column side face of $0.35d$.

![Diagram showing the arrangement of precast slabs and punching shear reinforcement in reference tests (notation O) and additional tests (notation E)]

a) with Filigran®-Punching Shear Reinforcement FDB II (FDB)

![Diagram showing the arrangement of double headed studs as punching shear reinforcement (DKA)]

b) with double headed studs (DKA)

Figure 7: Arrangement of precast slabs and punching shear reinforcement in reference tests (notation O) and additional tests (notation E)

Kueres et al.⁹ described two further tests with double headed studs function as punching shear reinforcement. In a reference test with a 285 mm thick composite slab, the precast slab part had a distance to the column of 40 mm (fig. 7b, left side). In a new test, the upper column surface was situated 20 mm above the lower surface of the slab and the precast part was placed flush with the column (fig. 7b, right side). The studs close to the edge in both arrangements had a distance to the column of $0.375d$. This was within the permissible range of $0.35d$ and $0.5d$.

All four punching shear tests were carried out at the same test facility. Figure 8 shows one half of each new test specimen and the test arrangement. In all four tests a 40 mm wide gap between the precast slab parts was poured together with the top layer of the composite slab. In both tests with Filigran®-Punching Shear Reinforcement FDB II the columns had square cross sections. Both specimens with double headed studs had columns with circular cross sections.
Figure 8: Test specimens and test arrangement of the additional test

Figure 9 shows the test specimen FDB-E from below. At some locations it was possible to slide in a plastic card and an annular clearance became obvious.

Figure 9: Test specimen FDB-E from below with a gap between column and precast slab

At the beginning of each test, the load was increased up to the service load, which was taken as the expected failure load divided by 2.1. The load was then cycled 10 times between this level and 50% of this level. Afterwards the load was increased incrementally to the maximum load. Figure 10 shows the load-deflection curve of the tests. The initial load $V$ was related to the expected characteristic load $V_{Rk,c}$ according to equation (1) to allow for easy comparison of the tests. The deflection was taken as the average of the deflections measured at four points at the circle of loading (fig. 8).

In both tests with the Filigran®-Punching Shear Reinforcement it was possibly, following a reduction of the load, to increase the load again to about 80% of the maximum load. This demonstrates the
ductile behaviour of the column slab connection. Both load deflection curves are quite similar (fig. 10, right side). No effect of the penetration of the column could be seen.

The load-deflection-curves of the tests with double headed studs (fig. 10, left side) deviate above load levels of 1.5 times of the resistance of a slab without shear reinforcement. A reloading of the test specimen DKA-E after reaching the maximum load was not done due to excessive deflection of the specimen.

![Figure 10: Load-deflection-curve of the punching shear tests compared](image)

After the tests the specimens were cut in the plane of the side surface of the column to study the internal crack pattern (fig. 11). Test specimens with double headed studs show inclined shear cracks typical for this type of reinforcement. A steep shear crack close to the column can be observed in test DKA-E as an initial failure crack. The crack tip points to the upper edge of the column concreted 20 mm too high. In the reference test DKA-O without column penetration the crack tip sits lower.

The crack pattern of tests FDB-O and FDB-E with Filigran®-Punching Shear Reinforcement FDB II show similar characteristic. In the sectional plane parallel to the longitudinal direction of the shear reinforcement only some cracks, which combine in the layer of the upper bending reinforcement can be observed. The sectional plane perpendicular to the above-mentioned one shows fine distributed cracks. A steep crack close to the column, which is not penetrated by shear reinforcement is not observed. This is determined by the reinforcing system, which was placed close to the column at a distance smaller than 0.35d (fig. 7a).

In every test specimen a horizontal crack in the compressive zone approximately in the middle of the precast part of the slab can be observed. Such cracks in the compression zone are also observed in other punching shear tests with high shear capacity.
Table 1 lists key parameters and results of the described tests. The achieved punching shear resistance is compared to the expected resistance of a slab with the same geometry and concrete strength but without shear reinforcement.

### 3.3 Results and Evaluation

Test FDB-E with the Filigran®-Punching Shear Reinforcement FDB II and an initial column penetration of 20 mm achieved an increase factor of $\alpha_{\text{max},i} = 2.20$ compared to the resistance of a slab without shear reinforcement. This value is about 2% higher than the compared value $\alpha_{\text{max},i} = 2.16$ in the test without column penetration. According to the assumption shown in figure 6, column penetration reduces the effective depth as well as the design perimeter. With the geometry of the tested specimen a reduction of the punching shear resistance of about 15% had been expected, which translates to an expected increase factor $\alpha_{\text{max},i} = 1.84$ in relation to the reference test.
Therefore, the test with the Filigran®-Punching Shear Reinforcement did not confirm the model according figure 6. The punching shear resistance was not reduced by the initial column penetration and was covered well by equation (3).

Table 1: Parameter and results of the punching shear tests

<table>
<thead>
<tr>
<th>Test notation</th>
<th>Double headed studs</th>
<th>Filigran®-Punching Shear Reinforcement</th>
<th>FDB II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference</td>
<td>DKA-O</td>
<td>DKA-E</td>
<td>FDB-O</td>
</tr>
<tr>
<td>Distance between precast slab and column</td>
<td>40 mm</td>
<td>0 mm</td>
<td>20 mm</td>
</tr>
<tr>
<td>Penetration of column</td>
<td>0 mm</td>
<td>20 mm</td>
<td>0 mm</td>
</tr>
<tr>
<td>Effective depth d</td>
<td>250 mm</td>
<td>250 mm</td>
<td>209 mm</td>
</tr>
<tr>
<td>Column cross section</td>
<td>Ø 263 mm</td>
<td>300 mm x 300 mm</td>
<td></td>
</tr>
<tr>
<td>Concrete cylinder strength top layer $f_{cm,cyl}$</td>
<td>29.9 MPa</td>
<td>31.2 MPa</td>
<td>21.9 MPa</td>
</tr>
<tr>
<td>Concrete cylinder strength precast part $f_{cm,cyl}$</td>
<td>40.7 MPa</td>
<td>42.3 MPa</td>
<td>22.7 MPa</td>
</tr>
<tr>
<td>Longitudinal reinforcement ratio $\rho_l$</td>
<td>1.25%</td>
<td>1.25%</td>
<td>0.70%</td>
</tr>
<tr>
<td>Expected resistance of a slab without shear reinforcement $V_{Rk,c}$</td>
<td>1003 kN</td>
<td>1020 kN</td>
<td>661 kN</td>
</tr>
<tr>
<td>Maximum load $V_u$</td>
<td>2085 kN</td>
<td>1975 kN</td>
<td>1428 kN</td>
</tr>
<tr>
<td>Resistance increase factor in test $\alpha_{max,i} = V_u / V_{Rk,c}$</td>
<td>2.08</td>
<td>1.94</td>
<td>2.16</td>
</tr>
<tr>
<td>Approved increase factor $\alpha_{max}$</td>
<td>1.96</td>
<td>2.09</td>
<td></td>
</tr>
</tbody>
</table>

$^a$ according to equation (1) taking into account $f_{ck} = f_{cm,cyl} - 4$ MPa

$^b$ Press load and dead load

The estimated increase factor is shown in figure 12 together with other test results. In these tests the distance of the precast part to the column edge was varied and the lower surface of the precast columns was situated on the level of the lower slab surface. All achieved results were - while scattered - in the range above the approved increase factor of $\alpha_{max} = 2.09$, which was determined in tests with Filigran®-Punching Shear Reinforcement in composite slabs.

Test DKA-E with double headed studs and an initial column penetration of 20 mm achieved an increase factor relative to the expected resistance of a slab without shear reinforcement of $\alpha_{max,i} = 1.94$. This figure is about 7% lower than $\alpha_{max,i} = 2.08$, which was estimated in the reference test without column penetration. According to the assumption shown in figure 6, a reduction of the punching shear resistance of 14% or a factor of $\alpha_{max,i} = 1.79$ respectively had been assumed. The achieved resistance in the test was therefore in between the comparative test result without column penetration and the assumed resistance taking into account a reduced effective depth.
3.4 Recommendations

In the test with the Filigran®-Punching Shear Reinforcement FDB II and an initial column penetration of 20 mm a shear resistance higher than the expected and approved at a value of $\alpha_{\text{max}} = 2.09$ was reached. This reinforcement system appears robust against small deviations from the planned height. This finding is supported by the fact that the approved increase factor was evaluated in tests with composite slabs with pre-existing vertical bond interfaces close to the column (figure 3). Therefore, it can be endorsed neglecting minor initial column penetrations in calculations when using this reinforcement according the approved application.

On the other hand, the single test with double headed studs showed a reducing effect from an initial column penetration on the shear resistance. This suggests that it is prudent to take an initial column penetration into account by reducing the effective depth when calculating the punching shear resistance of the slab.

As far as further tests offer additional information about the effect of initial column penetration into the slab Kueres et al. recommended situating the upper level of a precast column below or on the level of the upper surface of the slab in any case. In case of a higher concreted column it is recommended to calculate the punching shear resistance with a reduced effective depth. This can also be done in cases of higher column penetration than tested.

4 Conclusion

The Filigran®-Punching Shear Reinforcement FDB II was developed to increase the punching shear resistance of flat slabs. The maximum shear resistance of 2.09 times the resistance of a slab without shear reinforcement was determined as a characteristic value in tests with composite slabs. In these tests joints between precast elements as well as between the precast part and the column were tested. Despite these boundary conditions it raised the question whether an initial column penetration reduces the effective height of the slab and therefore its shear resistance.
A full scale test with a column penetration of 20 mm into a 260 mm thick slab was carried out and compared to a previous test carried out with the same dimension and the same test device. The penetration showed no effect on the load bearing behaviour. An additional test with double headed studs showed a slight decrease of the resistance in case of column penetration. This test and limited other investigations lead to the recommendation to precast a column not higher than the lower surface level of the slab or to calculate the resistance with a reduced effective slab depth.

References:


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8. ACI 318-14: Building Code Requirements for Structural Concrete. Reported by ACI Committee 318. American Concrete Institute (ACI), September 2014.

TEST METHODS
ON THE EOTA CORROSION RESISTANCE TEST OF POST-INSTALLED REBAR CONNECTIONS

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ABSTRACT

Requirements for the approval of post-installed rebars situated in concrete are described in EOTA Technical Report 0231. Within the document “Methods of verification” and the “Assessing and judging the fitness of anchors for an intended use” including the “Corrosion resistance of rebar” are indicated. The following article addresses the latter. Questions arose regarding the test procedure and those offered an incentive to investigate different factors of the corrosion test. For that purpose specimens with cast-in-place and post-installed rebars were investigated. A standard test according to TR 023 was performed. Furthermore, during a three-month period the open circuit potentials (OCP) of the working and counter electrodes were measured to gain information concerning influencing factors e.g. oxygen content. Results obtained were analyzed and compared to investigations in literature to give more information on the test procedure.

1 Introduction

Reinforcing steel bars (rebars) covered with concrete are protected against corrosion by the high alkalinity of the concrete’s pore solution with pH values above 12.6. In case of post-installed rebars there is no contact between corrosion protective concrete and rebar. Therefore, the adhesive mortar has to adopt the corrosion protective function from the concrete. For post-installed rebars, consisting of adhesive mortar and rebar, situated in concrete, which exceed exposure classes X0, and XC1 according to DIN EN 1992-1-1 it has to be shown that they provide the same corrosion resistance as cast-in-place rebars1. The EOTA Technical Report 0231 chapter 3.3.4 comprises the test procedure for “Corrosion resistance of rebar”.

The EOTA TR 023 prequalification procedures require for the verification of corrosion protection, that the post-installed rebar systems are placed in chloride contaminated concrete with chloride contents (expressed as mass proportion of chloride ions in cement) of 0.2 % or 0.4 % for category 1 or 2, respectively. The specimens are partly placed in artificial tap water for three months with instrumentations to measure corrosion potential and corrosion current between a stainless steel counter electrode and the rebar. If potential and/ or current within the last third of the test period adhered to given limits and no corrosion appeared at the rebar surface the test passed.

At the Materials Testing Institute (MPA) University of Stuttgart, numerous corrosion tests have been conducted with different post-installed rebar systems from different manufacturers according to
EOTA TR 023. Normally tests at different mortar systems either obviously passed the procedure or failed, see Figure 1. However, there were some tests with no distinctive result. Some tests showed corrosion currents beyond the current limits but only for a short time. Some specimens showed minor corrosion spots, which were located only at the bottom saw cut of the rebar. In addition, tests with cast-in-place rebars indicated corrosion at the transition between concrete and air. Some but not all plots with results of potential and current reflected this corrosion. These “grey” areas deviate from the black or white / yes or no requirements of the assessment procedure and gave reason to have a closer look at the influences on the “Corrosion resistance of rebar” as stated in EOTA TR 023\(^1\).

![Figure 1: Post-installed-rebars after 3-month test period without corrosion, test passed (left) and with corrosion, test failed (right)](image)

2 Background – test description

By means of the test of “Corrosion resistance of rebar” within EOTA TR 023 it shall be proved that the used mortar accounts for the corrosion protection of the rebar equivalent to a concrete environment. The test setup is shown in Figure 2.

![Figure 2: Schematic sketch of test setup. Source: EOTA Technical Report 023, Nov. 2006](image)
The detailed test procedure is described in EOTA TR 023. At this point only the relevant facts shall be described. The tests are performed on concrete cubes or beams. The requirements for the concrete mixture are described in ETAG 001, Appendix A. However, the w/c-ratio is limited to 0.6. This value yields confusion since it is not annotated whether the limitation is meant as lower or upper border. There is a precise description in AC308, covering the prequalification of post-installed rebars in the USA, which states: “The water-cement ratio used for the mix design shall be equal to or greater than 0.6.” A variety of three different cement types might be used: (i) CEM I (ii) CEM II A/LL or (iii) CEM II B/LL. The concrete strength shall match the strength class C 20/25.

The concrete samples will be immersed into a container filled with artificial tap water. The water level is kept 90 mm above the bottom side of the concrete samples. Evaporated water is refilled by distilled water to keep a constant waterline and a constant ion concentration. Each rebar is connected to a cathode with a 100 Ohm resistor (accuracy class ± 1%). The cathodes are L-shaped and made of stainless steel (EN 10088 1.4404). They are positioned directly on the bottom of the container. The surface of the cathodes in contact to the water is at least 100 cm². The corrosion potential is measured between the rebar (working electrode) and the reference electrode. The current between the rebar and the cathode is readily determined by measuring the potential drop over the resistor of each rebar. For the measurements, the micro voltmeter Keithley 2701 with a resolution of 0.1 µV and an input resistance > 10 GΩ was used.

EOTA TR 023 is very precise in its requirements regarding the assessment of test results, which are stated below:

(a) During the last third of the testing period the daily mean value of the current shall not exceed 0.28 µA and the potential shall not be below -0.2 V CSE for all test samples.

(b) The potential criterion may be omitted if the current criterion of 0.28 µA is fulfilled for all samples and the visual inspection of the rebar after the test does not show any corrosion products.

If either condition (a) or (b) is fulfilled the corrosion resistance of the post-installed rebar connection can be judged as being comparable with the corrosion resistance of cast-in-place rebar.

On the other hand, the requirements on specimens, especially options on concrete (“w/c-ratio is limited to 0.6”, three different cement types are possible) and steel (no requirements on the steel-type of rebar) as well as the testing period (“at least 3 months”) are not precise. Due to rigorous requirements on current limits of 0.28 µA, the test is extremely sensitive to any galvanic corrosion process (between working electrode and counter electrode) and other effects (redox system, diffusion effects) taking place on the rebar surface inside the specimen or the concrete. It might be indicated that 0.28 µA related to an embedded surface area of around 27.5 cm² (diameter 12 mm and embedment length of 70 mm) results in a current density limit of around 0.01 µA/cm².

### 3 Experimental research

Aim of the presented investigations was the presentation of typical results of the test method as well as factors, influencing the rest results. Therefore, a standard TR 023 test with four specimens (two
cast-in and post-installed) was performed. Furthermore, the open-circuit-potential (OCP) of the working electrode (rebar) and the counter electrode (stainless steel plate) was measured at two specimens over the standard testing period of three months. The type of mortar was accidentally chosen.

### 3.1 Test samples

The tests were performed at concrete cubes with an edge length of 150 mm. The cubes were produced at MPA laboratories with concrete mixture according to ETAG 001, Appendix A². The concrete had a w/c ratio of 0.65, cement type CEM I with a content of 260 kg/m³ and corresponded to strength class C20/25. At some samples, a chloride content of 0.4 % per weight of cement with aid of sodium chloride was added. All samples were cast in December 2015. After production, the cubes were stored at 20°C and 100 % RH for 7 days and at 20°C and 65 % RH for 21 days, according to DIN EN 12390-2, Appendix NA⁵. After curing of 28 days, the post-installed rebars were installed strict according to the manufacturer’s installation instruction. The rebar samples correspond to a B500B, hot rolled and cold strained, according to DIN 488-1:2009-08. To prevent carbonation a layer of epoxy resin was applied at the concrete surface around the rebar. The samples with chloride content were stored in artificial tap water according to TR 023¹ until testing in February 2017. The samples without chloride content were stored at room climate of around 20°C and 50 % RH. For the tests, the concrete samples were immersed into a container filled with artificial tap water (200 mg sodium sulphate and 200 mg sodium bicarbonate dissolved in 1 liter distilled water). By means of plastic spacer, the concrete samples were kept 10 mm above the bottom of the container. The water level was kept 90 mm above the bottom side of the concrete samples. Evaporated water was refilled by distilled water to keep a constant waterline and constant ion concentration over the duration of the tests.

#### Table 1: Samples tested

<table>
<thead>
<tr>
<th>TR 023 samples – Rebar tests (RT)</th>
<th>label</th>
<th>type</th>
<th>chloride</th>
<th>stored</th>
<th>remark</th>
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</thead>
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<tr>
<td>Ref-castin</td>
<td>cast-in</td>
<td>-</td>
<td>air</td>
<td></td>
<td></td>
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<tr>
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<td>0.4 %/wt.</td>
<td>water</td>
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<td></td>
</tr>
<tr>
<td>0.4Cl-castin_2</td>
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<td>0.4 %/wt.</td>
<td>water</td>
<td>WE and CE measurements</td>
<td></td>
</tr>
<tr>
<td>Ref-mortar</td>
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<td>-</td>
<td>air</td>
<td></td>
<td></td>
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<tr>
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<td>0.4 %/wt.</td>
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<td>mortar</td>
<td>0.4 %/wt.</td>
<td>water</td>
<td>WE and CE measurements</td>
<td></td>
</tr>
</tbody>
</table>

WE – working electrode, CE – counter electrode

### 3.2 Rebar tests according to TR 023

The corrosion resistance of rebars according to TR 023 was tested by connecting each rebar to a cathode with a 100 Ohm resistor (accuracy class ± 1%). The cathodes are L-shaped and made of stainless steel (EN 10088 1.4404). They were positioned directly on the bottom of the container. The
surface of the cathodes in contact to the water was at least 100 cm². Previous to the test, the cathodes were degreased with ethanol, cleaned by exposing for 10 min to 5 % Nitric acid and subsequent rinsing with distilled water. The cathodes have been stored in the artificial tap water for at least 2 weeks, prior to the start of the test. The current between the rebar and the cathode was readily determined by measuring the potential drop over the resistor with the micro voltmeter Keithley 2701 with a resolution of 0.1 µV and an input resistance > 10 GΩ.

Additionally, the corrosion potential of each rebar was measured against a reference electrode in the container. As reference electrodes Ag/AgCl-electrodes (3 M KCl) were used. The results of potentials measured are related to saturated Cu/CuSO₄-electrodes (CSE). The difference between Ag/AgCl-electrodes and saturated Cu/CuSO₄-electrodes (CSE) is accounted for by -113 mV. The measurement of the current flow and the potential were done continuously in intervals of 10 minutes.

Table 2: Half cell potentials of selected reference electrodes

<table>
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<th>Ref. electrode electrolyte</th>
<th>Potential vs. SHE T = 25°C [mV]</th>
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<td>SHE</td>
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<tr>
<td>CSE</td>
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<td>+320</td>
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<tr>
<td>Ag/AgCl</td>
<td>3 M KCl</td>
<td>+207</td>
</tr>
</tbody>
</table>

Figure 3: Test setup for TR 023 corrosion resistance of rebar test

4 Influences ascertained by test results and literature study

4.1 Overview

The influences on the test results are graphically shown in Figure 4. There are three groups of influencing factors: (i) test conditions, (ii) concrete and (iii) installation (including rebar preparation). Each group and further each factor may or may not affect the different sample types (cast-in-place
and post-installed). A detailed presentation of test results and facts from literature regarding the influences are presented in the following.

4.2 Influences due to test conditions

The daily mean values of potential and current of four regular corrosion tests and two samples with measured open circuit potentials of working and counter electrode over a period of around 95 days are shown in Figure 5. The low potential and high current of sample Ref-castin will be described in the following section. At this point a characterization of the potential curves of corrosion tests and open electrodes is presented.

Figure 5: a) Corrosion potential of four rebar tests (bold lines) and open circuit potential of working electrode (WE) and counter electrode (CE) of two samples (thin lines) and b) Corrosion current of four rebar tests
The potential curves of corrosion tests and working electrodes show abrupt drops in trend. Contrary to that, the counter electrodes show an abrupt rise at the same time. It was found that this behavior is caused by the water refill necessary due to evaporation. During testing period water refill was conducted around once a week. One exception was at day 60. There is an exceptional strong rise of the potentials of the counter electrodes. The reason was that there were more than two weeks Easter holidays without refill. However, the rise of potential of the stainless steel counter electrodes was caused by increase of oxygen content in the artificial tab water due to fresh water and water movement. In Figure 5a it can be seen, that both counter electrodes show nearly the same behavior over the testing period. Compared to the working electrodes the potential of the counter electrodes show a sudden rise of around 9 mV, see Figure 6. The potential drop effect at working electrodes is smoother and due to diffusion effects at the concrete. Due to evaporation of water, the waterline at the samples drops down by 10 mm or 15 mm per week. Concrete pores dry out due to desorption of water. A sudden water refill leads to diffusion effects that will be measured as potential drops although there is no actual change at the working or counter electrode. Angst et al.6, 7 investigated that effect for concrete and mortar in more detail. However, the effects caused by water refill might have no significant influence on the state of passivity or state of corrosion of the rebar.

Figure 6: Detail of open circuit potentials of working and counter electrode at water refill of day 60, see Figure 5a

During the test period of around 95 days the temperature of the laboratory was monitored. There was a maximum delta of 5°C. However, this effect was not reflected by potential or current data.

4.3 Influences due to concrete

The influences of concrete might either be caused by carbonation or by different concrete mixtures and conditioning, respectively. However, these factors primarily affect cast-in-place rebars. One effect found during the investigations was corrosion of all cast-in rebars within the top region (around 5 mm to 10 mm) of concrete, see Figure 7. For one sample (Ref-castin) the corrosion effects were also depicted in potential and current trend, although this sample did not contain chlorides, see Figure 5. The 0.4Cl-castin_1 sample demonstrated the same corrosion appearance at the rebar but no indication on potential and current measurements. It can be assumed that the corrosion occurred and stopped before testing the sample since the samples were in contrast to regular testing practice more
than one year old at the beginning of the test. The corrosion at the top layer of concrete was not found at any tested mortar sample, see Figure 8. A carbonation test of a currently cracked sample revealed the reason of the corrosion. There is a ca. 5 mm layer of carbonated concrete at the top cast side of the sample, see Figure 9. Despite of the layer of epoxy resin at concrete, carbonation was found. That indicates a carbonation process before applying epoxy resin, which again means, carbonation took place within the four weeks conditioning time. The carbonation of the top layer is favored by a relatively high w/c-ratio of 0.65. It has to be at least 0.6. Furthermore, due to compaction the top layer of concrete might tend to segregation and accumulation of a higher water level which in fact results in a higher porosity and faster carbonation.

There was also one spot of corrosion of around 1 mm in diameter found on a rib at the cast-in rebar. This might be caused by a larger air void and is not related to the carbonation at the concrete surface.

Figure 7: Cast-in-place rebar after testing period with corrosion within top layer of concrete sample as well as one spot of corrosion on one rib

Figure 8: Post-installed rebar after testing period without corrosion

Figure 9: Carbonation test of concrete after 3-month testing period despite of epoxy resin cover
The concrete mix design proposed in ETAG 001\textsuperscript{2} allows a variety of different cement types, w/c-ratios and aggregates. Especially the first two factors and the type of conditioning might influence the level of the open circuit potential (OCP) of rebars. Beck\textsuperscript{8} investigated these factors in detail and found distinct influences on cement type, w/c-ratio and conditioning on the position of the open circuit potential (OCP) of steel in concrete, which is one criterion of the TR 023 test. By comparing CEM I to a CEM III/A concrete samples with cast-in steel bars a distinct increase in OCP and overall higher values of OCP for CEM I concrete compared to CEM III/A concrete were found. Different w/c-ratios of one concrete type showed lower OCP values for lower w/c-ratios. A conditioning with high relative humidity (95\%) showed lower OCP values as relative humidity of 50\% and 85\%. In fact, all factors lowering the oxygen content and diffusion at and to the working electrode, respectively, result in lower OCP, e.g.: lower w/c-ratios, conditioning under water or at high relative humidity and cement types resulting in a dense concrete with less capillary porosity.

4.4 Influences due to installation

There are two aspects summarized under this section: (i) differences in rebar surface and (ii) installation of rebar. The first aspect mainly intends the differences between the rebar’s lateral surface and the bottom surface, which is under test conditions typically a saw cut surface. The lateral surface is the “natural” rebar surface, degreased with ethanol. Misleadingly, EOTA TR 023\textsuperscript{1} says: The rebars “should be ensured to be free from mill scale and other loose contaminants ...”. However, the mill scale is the natural appearance – a thin layer of oxides at the surface which is not a loose contaminant. Furthermore, the removal of the mill scale requires pickling or mechanical cleaning, which both strongly changes the steel surface. In contrast to the lateral surface, the bottom surface is typically a saw cut with chamfered edges, see right hand pictures of Figure 7 and Figure 8. The corrosion kinetics of both surfaces are different and thus under the same conditions within the concrete sample or the mortar surrounding, both surfaces exhibit different corrosion behavior.

In addition to the aforementioned fact (i), the installation of the rebar (ii) for both, cast-in and post-installed, might result in a different phase boundary between rebar and concrete/ mortar of the lateral surface and the bottom surface. The compaction of concrete of samples with cast-in rebars might result in air voids underneath the rebar where corrosion is favored\textsuperscript{8}. For post-installed rebars EOTA TR 023\textsuperscript{1} says: “The rebar is positioned so as to rest on the bottom of the drilled hole”. Which means, there is a thinner mortar layer beneath the bottom surface than at the side surface. Of course, by inclined installation of the rebar, which is unpreventable in praxis, the mortar layer at the lateral surface might alternate as well. However, the rebar bottom in addition exhibits a different steel surface. Testing praxis showed that corrosion at the bottom surface is more likely than corrosion on the lateral surface.

5 Conclusion and suggestions on the EOTA TR 023 corrosion test

The EOTA TR 023 corrosion resistance test is a good procedure to examine the corrosion resistance of a mortar system for post-installed rebars. However, very precise requirements regarding the test results stay in contrast to a wider range of parameters regarding the test samples. In addition, some sample parameters may favor corrosion or at least lead to difficulties to interpret the results.
For that reason, the following clarifications or changes in test procedure could be a possibility to make the test procedure more precise and easier to interpret:

1. Coating the transition zone from air into concrete and the bottom zone of the rebar with a protective coating to prevent corrosion due to carbonation or as result of different rebar surfaces. In fact, a defined lateral testing surface is reached.

2. Specifying the concrete mixture with designation of w/c-ratio (e.g.: 0.6), cement type (e.g.: CEM II/A-LL) and conditioning procedure. Furthermore, specification of testing period in days, e.g.: 91 days (13 weeks).

3. To test three cast-in-place samples per test series to gain a reference for the applied rebars and other test parameters.

6 Acknowledgement

At this point, I would like to acknowledge those companies who supported this work by dedicating their specimens and permitting the print of pictures and test results. Particularly I would like to thank Dr. Klaus Menzel for the consulting support in terms of corrosion and Prof. Dr. Werner Fuchs for the consulting support in terms of background of EOTA Technical Reports, especially No. 023.

References:


THE BEAM END TEST AS A TEST SPECIMEN FOR THE BOND OF REINFORCEMENT BARS IN CONCRETE

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ABSTRACT

To guarantee high quality of concrete structures the bond properties between steel and concrete have to be kept within certain limits with respect to bond strength and bond slip. Therefore test specimens are used to proof the bond-slip-behavior of reinforcement bars. These tests have to reflect the bond behavior realistically and provide repeatable results. The current test specimens provide results with high variances and are only approved for static loading. With a modified Beam End Test, a new test beside the tests given in RILEM is introduced. It is cost effective and first tests showed favorable results concerning reproducibility and functionality on reinforcement bars with different rib geometries. The results show that the proposed test is able to show the differences in the bond-slip-behavior of different reinforcing bars depending on the related rib area. The behavior under static loading is described.

This paper shows the Beam End Test’s capability to describe the bond-slip-behavior, as well as a detailed description of the modified test specimen is given. As a conclusion of the results of static load tests, an assessment criterion for the Beam End Test is given.

1 Introduction

The bond properties between reinforcement bars and concrete are essential for the load bearing capacity of reinforced concrete. Reinforcement bars are provided with ribs. Thereby a mechanical interlocking between concrete and steel is achieved and bond stresses can be transferred. To ensure the load bearing capacity of reinforced concrete, the bond properties of the steel bars have to be kept at a high level. DIN 488-1\textsuperscript{3} contains nominal diameters and requirements for the surface geometry of steel bars. Dimensions, profile and composition of ribs are given as well as minimum values for the related rib area represented by the value $f_R$. Besides DIN 488 there are international codes regulating reinforcing bars (fib Model Code (2010)\textsuperscript{13}, EN 10080: 2005\textsuperscript{8}). Partially there are immense deviations concerning surface geometry and the limit for the related rib area $f_R$. DIN 488 contains the most limiting regulations in comparison to other regulations.
Alternatively, to ensure sufficient bond behavior related to the requirements of structural design, the bond properties can be tested according to DIN EN 10080\textsuperscript{8} to qualify a reinforcing bar with the RILEM Beam Test or the RILEM Pull-Out Test described in Ref.\textsuperscript{8}. This procedure is necessary if the requirements due to surface geometry or related rib area \( f_R \) are not fulfilled.

The problematic in testing the bond properties with these tests is that the results depend on which test (Pull-Out or Beam Test) is performed to qualify the reinforcing bar. Test results given in Ref.\textsuperscript{12} show that reinforcing bars may not be qualified even if they fulfill all requirements according to DIN 488 and on the other hand it is possible to qualify a steel reinforcing bar that does not fulfill the minimum values for \( f_R \). Underrunning the minimal related rib area however could influence the serviceability (with respect to the crack widths) and safety of the building in a negative way. So it is mandatory that a qualification test with reasonable acceptance criteria for bond properties is necessary. This test must represent the bond behavior of reinforcement bars realistically and in a reproducible way.

In Ref.\textsuperscript{17} therefore a modified version of the Beam End Specimen used for the qualification of reinforcing bars according to ASTM standard\textsuperscript{1} is recommended. In the presented test series the results of the Beam End Test were compared with the results of the Beam Test and the Pull-Out Test. The aim of the research was to show that the Beam End test is practical for standardization and more reliable as well.

In the test series given in Ref.\textsuperscript{12} the following parameters were varied:

- nominal diameter \( d_s \),
- related ribbed area \( f_R \) and
- anchorage length \( l_b \) (only Beam Test and Pull Out Test).

In addition to the experiments, a finite element model was used to demonstrate the effect of different bond-slip behaviors at a specimen level and a structural level. The model was calibrated against the experiments and used for further investigation of the influence of \( f_R \) and rib geometry. The evaluation of experiments and calculations showed up a deficit of the current situation because there were steel bars passing one test and failing in the other. So the results clearly show that the different tests are not consistent and will come up with different results and assessment of the reinforcing bars. To overcome this contradiction in Ref.\textsuperscript{12} the Beam End Test is recommended for the proof of reinforcing bars. However until now there are no criteria for the qualification given and the criteria given in Ref.\textsuperscript{12} are based on only a few tests and do not consider the scatter of the bond slip behavior.

Since only a few tests were made there are still a lot of open questions to be answered. The test series that were started in Ref.\textsuperscript{17} have been continued at the Institute of Construction Materials at the University of Stuttgart in 2012. The investigations done in 2012 contain tests under static loading with the focus on different rib geometries available in the market as well as tests under cyclic loading with the focus on the detrition of bond as a function of the number of load cycles. The modified test specimen, the test procedure and the results are presented in the following sections.
2 Basic information

2.1 Bond stress

Concerning the quantification of bond in concrete, the significant parameters are the bond stress and the corresponding slip at different stress levels. By loading the reinforcement bar, bond stress between the reinforcing bar and surrounding concrete is activated. For the bond situation given by the Beam End Test, the mean bond stress can be calculated according to Equation (1) assuming a uniform bond distribution over the tested bond length. The assumption of a uniform bond stress is assumed to be valid due to the fact that the bond length is small enough.

\[
\tau = \frac{F}{\pi \cdot d_s \cdot l_b}
\]  

\( \tau \) bond stress  
\( F \) applied load in axial direction of reinforcement bar  
\( d_s \) nominal diameter of reinforcement bar  
\( l_b \) bond length of steel in test member

Subsequently \( \tau_{\text{max}} \) stands for the maximum bond stress. This maximum bond stress corresponds to the bond stress measured at maximum tested load \( F_{\text{max}} \). In the context of this article also the abbreviation \( \tau_{0.1} \) is used. This value represent the bond stress measured at a slip value of \( s=0.1\text{mm} \). The slip is defined as the displacement of the reinforcing steel bar relative to the concrete member measured at the end of the bond length (unloaded end). The slip therefore does not include the deformation of steel or concrete over the bond length.

2.2 Related rib area \( f_R \)

The bond stress that can be activated is significantly influenced by the rib geometry of the reinforcing steel bar. Increasing the mechanical interlock between concrete and reinforcing bar will also increase the bond stress that can be activated while loading. To represent the surface geometry of a reinforcing bar in one parameter, the related rib area \( f_R \) was defined\(^1\). The related rib area incorporates the relation between projected rib area and lateral area of the steel bar (see Figure 1). According to Ref.\(^{15} \) the related rib area can be calculated with Equation (2).

![Figure 1: Related rib area\(^{17} \)]
\[ f_R = \frac{F_R}{F_s} = \frac{\pi \cdot (d_k + a) \cdot a}{\pi \cdot d_k \cdot c} \approx a \]

However it should be mentioned that the bond-slip behavior depends mainly on the geometry of the ribs and not on the related rib area. This is due to the fact that very different geometries could provide the same related rib area.

3 **Acceptance criteria for reinforcement bars**

With the RILEM Pull-Out Test and Beam Test, two tests for the qualification of reinforcement steel bars given. In the German codes only requirements for the rib geometry are given. According to Ref.\textsuperscript{5} it is mandatory that cold-formed reinforcing bars have 3 rows of cross ribs. They have to be crescent-shaped and the angle between rib and bar axis should be in between 40 and 70 degrees or between 35 and 75 degrees if the angles are alternating. The required dimensions of the ribs are given in table 4 in Ref.\textsuperscript{5}. For reinforcing bars with a diameter between \( d_s = 11 \text{mm} \) and \( d_s = 40 \text{mm} \) a 5%-quantile of \( f_R = 0.056 \) is given. Apart from that no criterions for the qualification with experiments are available. In the literature, criteria for the Beam Test\textsuperscript{2} and only a proposal for the Pull-Out Test\textsuperscript{12} can be found. These criteria consider the bond stress at maximum load and at a slip value of \( s = 0.01 \text{mm} \) and \( s = 0.1 \text{mm} \). Due to the fact that both tests will come up with a different assessment and the criteria are not consistent, it is questionable whether they can be used for standardization.

New criteria are proposed for the Beam End Test in Ref.\textsuperscript{17}. These criteria must be fulfilled in the tests and represent the minimum bond strength values for reinforcing steel bars that will fulfill the requirements on the surface geometry and the related rib area. The values of two points on the bond-slip-curve are considered: The maximum bond stress \( \tau_{\max} \) and the bond stress measured at a slip value of \( s = 0.1 \text{mm} \) \( \tau_{0.1} \). If the measured bond stress in the test is higher than these values, the steel can be assumed to be qualified. Since the criteria are dependent on the nominal diameter they can only be applied for steel bars with \( d_s \) not greater than \( d_s = 25 \text{mm} \). The requirements for the bond stress according to Ref.\textsuperscript{17} are given with the following equations:

\[ \tau_{\max} \geq 11.33 - 0.17 \cdot d_s \]  
\[ \tau_{\max} \geq 11.57 - 0.28 \cdot d_s \]

Whether these criteria are suitable for the standardization of reinforcement bars will be discussed with the test results given in the following section.
4 Experimental procedures

4.1 Test specimen

The experiments in Ref.\textsuperscript{17} show, that the Beam End Test has a lot of advantages in comparison to the Beam Test with respect to the manufacturing, testing procedure, reproducibility of test results and therefore also due to economic efficiency within a qualification process of reinforcing bars. To show that the Beam End Test can be used to qualify reinforcing steel bars and to develop the acceptance criteria, tests with different reinforcing bars were performed. To ensure that the results are comparable, all tests were carried out with the same Beam End Test specimen shown in Figure 2.

The used specimen in Ref.\textsuperscript{11} was modified compared to the test specimen in Ref.\textsuperscript{17} in a way that it was possible to perform two tests in one specimen. Therefore the specimen was designed diagonally-symmetric so the second test can be done just by turning the specimen around. Details on the test specimens can be found in Ref.\textsuperscript{11}, Ref.\textsuperscript{17} and Ref.\textsuperscript{18}.

![Test setup in Ref.\textsuperscript{11}](image1)

![Test setup in Ref.\textsuperscript{17,18}](image2)

Figure 2: Test specimen for Beam End Test

4.2 Reinforcement bars and test program

The tests were carried out with diameter between $d_s = 10$ mm and 50 mm. The aim of the performed tests was to check if the Beam End Test is able to separate between different qualities (in terms of rib geometry) of reinforcing steel bars. For that purpose three types of different steel reinforcing bars were chosen with different shape of the ribs and with a different related rib area. In the context of this article they are designated as B500B+P, B500A and Epstal. The technical data of the reinforcing bars as well as the details are given in Table 1.
Table 1: Evaluated tests - details

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<th>Ref.</th>
<th>Series</th>
<th>n</th>
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<td></td>
<td></td>
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1) Imprint „Epstal“ within the bond length
2) Imprint „Epstal“ outside the bond length
3) $f_R$ realized by polishing the ribs
4) debonded length: 10 $d_s$

5 Discussion of experimental results

5.1 Influence of concrete strength

In general, concrete strength is assumed to have an influence on the bond strength that can be covered by a square root function. The analysis of the experimental results achieved in Ref.11 and Ref.17 shows that the test results indicate a power function with an exponent of roughly 0.5 for the maximum bond strength, if the influence of the ribbed area is not considered. However there were a lot of tests performed with different concrete strengths, different $f_R$-values and different diameters. Most authors came to the conclusion that the influence of the concrete strength is between 0.5 and 0.66. If only the tests given in Ref.11 are evaluated the influence of the concrete strength is up to a power of 0.67.
Figure 2: Maximum bond stress $\tau_{\text{max}}$ (a) and bond stress $\tau_{0.1}$ (b) as a function of concrete cube strength $f_{\text{cc,150}}$

In Figure 2, the maximum values $\tau_{\text{max}}$ and values $\tau_{0.1}$ (bond stress at a slip of $s=0.1\,\text{mm}$) are plotted as a function of the concrete strength for each test.

The black squares represent the maximum bond stress $\tau_{\text{max}}$ (Fig. a) and $\tau_{0.1}$ (Fig. b). The gray marks are not considered due to the fact that the related rib area was realized by polishing the ribs of a normal reinforcing bar to achieve a $f_R=0.02$ and additional tests have been performed with a higher debonded pre-length ($10\,d_s$ instead of $5\,d_s$). The results of these tests are not consistent with the other results. All other related rib areas were between $f_R=0.07$ to $f_R=0.10$ beside the BST 500B(+P) with a $f_R = 0.036$.

Regarding the correlation between the results and the given trend lines, it is obvious that the relationship with a square root function expressed by the exponent 0.5 does fit for $\tau_{\text{max}}$. The given trend lines also show a higher exponent up to 0.95 for $\tau_{0.1}$. If only the test series given in Ref. 11 are considered the correlation between concrete strength and bond stress according to Ref. 12 with the exponent of $0.66 \approx 2/3$ is verified. So the results mainly depend on which results are considered and which diameter or related rib areas are looked on. Due to a lack of test results to figure out the influence of the concrete strength more precisely, the square root function with an exponent of 0.5 given in the Model Code 2010 is used for the normalization of the test results. However the results performed show a clear indication that the concrete strength is underestimated especially for bond values below $\tau_{\text{max}}$.

5.2 Influence of the bar diameter

The influence of the diameter was investigated in Ref. 17 for a related rib area of roughly $f_R = 0.07$. The results show that the maximum bond strength is decreasing with an increasing diameter. For the evaluation, only rebars with a rib area between 0.07 and 0.1 are used. The rebars with a rib area of 0.02 (polished) and 0.036 (steel wire) as well as one test series with longer debonded length are not considered. The results therefore show that the influence of the bar diameter can be normalized using a power function with a negative exponent (-0.15).

This effect is well known and in most cases independent of the level of bond strength. This can also be seen in the tests given in Ref. 12 if the influence of the diameter is plotted for $\tau_{\text{max}}$ and $\tau_{0.1}$. In
Figure 4 the measured bond stress $\tau_{\text{max}}$ and $\tau_{0.1}$ (normalized to a concrete strength of $f_{cc,150} = 25$ N/mm$^2$) are given as a function of the bar diameter. The influence is roughly the same for $\tau_{\text{max}}$ and $\tau_{0.1}$.

The result on the influence of the diameter on $\tau_{\text{max}}$ and $\tau_{0.1}$ is within the expected range of bonded rebar anchors or post installed rebar. In the following, all test results are normalized to a diameter $d_s = 14$ mm. The tests with a rib area of $f_R = 0.02$ and the tests performed with a higher debonded length are not evaluated.

5.3 Influence of related rib area $f_R$

According to the given literature Ref.15 and Ref.12 the maximum bond strength increases if the related rib area is larger. According to Ref.12 the bond slip behavior can be estimated as a square root function of the slip $s$, the concrete strength $f_c$ and the related rib area $f_R$. In the basic formulation of Ref.12 the bond strength is a linear function of the related rib area (Equation 5).

$$\tau_{\text{max}} = 10 \cdot \sqrt{s} \cdot \left( \frac{a}{c} \right) \cdot f_c$$

(5)

The maximum bond stress increases with higher values and decreases with lower values for $f_R$. In order to limit the minimum bond stress, DIN 488 provides lower bound values for $f_R$ to ensure a minimum bond stress at a certain slip value and a minimum bond strength $\tau_{\text{max}}$.

By definition, B500B+P is a reinforcement steel wire instead of a reinforcement steel bar. So it does fulfill this requirement and provides only a related rib area of $f_R=0.036$. This can be seen in Figure 4a were the bond strength $\tau_{\text{max}}$ of the tests is plotted as a function of the related rib area $f_R$. Also the bond stresses at a slip of 0.1 mm ($\tau_{0.1}$) are plotted over the corresponding values for $f_R$ (Figure 4b). The relation between related rib area and the measured bond stresses $\tau_{\text{max}}$ and $\tau_{0.1}$ indicates that the bond stress depends linearly on $f_R$.

Due to the fact that rebar with a related rib area $f_R=0.056$ were not available for tests, the bond stresses can only be interpolated to come up with the bond stress values for the minimum allowable related rib area $f_R$. Due to the fact that $f_R=0.056$ is the 5%-quantile given in DIN 488 for $d_s=14$ mm.
the requirement must be converted to a bond criteria using the bond stresses ($\tau_{\text{slip}}$ and $\tau_{\text{max}}$) for a $f_R$ value of 0.056.

To find such a criterion, the tests results are normalized to a concrete strength of 25 N/mm$^2$ using a square root function. In addition, all the test results are normalized to a diameter of $d_s = 14$ mm to show the influence of the related rib area without having other influencing parameters varied at the same time. The results are given in Figure 5a ($\tau_{\text{max}}$) and Figure 5b ($\tau_{0.1}$).

The results shown as grey dots are the results of the tests on rebar with $f_R = 0.02$ (realized by polishing a standard rebar) and the rebar tested with larger debonded length. These results are not taken into account for the evaluation. However for the maximum bond strength the results of the rebar with $f_R = 0.02$ fits well in the evaluated linear trend. For a bond stress at a slip of 0.1 mm ($\tau_{0.1}$) the values are much higher than expected. This is explained by the fact that the height of the ribs was reduced by polishing them and therefore besides the height the distance of the ribs was unchanged. For small slip values therefore the stiffness could be much larger. In addition, it has to be considered that the rib geometry was undefined by realizing that the rib height is not identical over the entire bond length or diameter due to the mechanical preparation$^{17}$.

From the test results it seems reasonable to assume a linear function between the bond stresses $\tau_{\text{slip}}$ and the related rib area $f_R$. The linear function therefore is used to come up with mean bond stress for a related ribbed area of 0.056 that is the minimum requirement according to DIN 488$^5$.

### 6 Assessment criteria for Beam End Test

To derive a criteria for the BET (Beam End Test), the bond stresses at a slip value of 0.1mm and the maximum bond strength ($\tau_{\text{max}}$) are evaluated at a related rib area of $f_R = 0.056$ using a linear interpolation.
Figure 5: Maximum bond stress $\tau_{\text{max},25,d=14\text{mm}}$ and as $\tau_{0.1,25,d=14\text{mm}}$ as a function of the related rib area $f_R$ and the corresponding mean values for $f_R = 0.056$, all tests given in Ref.\textsuperscript{11} and Ref.\textsuperscript{17}

The results show that the mean value of the bond strength ($\tau_{\text{max}}$) in the bond tests with a BET shall be at least 8.9 N/mm$^2$ and at a slip value of 0.1 mm the bond stress ($\tau_{0.1}$) shall be at least 5.8 N/mm$^2$.

A criterion that is based on the mean level neglects the scatter of the tests results. This can be solved by providing a 5% fractile value that includes the mean value and the standard deviation (but depends on the number of test results) or by providing a maximum scatter that is allowed in the tests. To come up with reasonable values, the coefficient of variation (COV) of the tests results was evaluated for the reinforcement steel bar B500A.

Figure 6: Evaluated coefficient of variation for different bond stresses at a define slip value ($\tau_{s,25,d=14\text{mm}}$), only tests given in Ref.\textsuperscript{11} with B500A

The test results show a significant increase of the coefficient of variation (COV) for smaller slip values. The COV is less than 5% for the bond strength ($\tau_{\text{max}}$) and the bond stress at a slip of 0.5mm. For a slip value of 0.1mm and 0.05mm the COV is 10% and 20% respectively. For a small slip value of 0.01mm the scatter increases and is larger than 30%. If it is assumed that the concrete properties will mainly influence the scatter of the bond stresses they should be in a range up to 15% to 20%. The results also show that the COV does not strongly depend on the related rib area $f_R$. A larger scatter should be avoided also due to the fact that the partial safety factor $\gamma_{\text{Mp}}$ is related to the scatter of properties in this range.
In the following, the criteria for the BET are derived to ensure the minimum requirements for reinforcing steel bars with respect to the initial stiffness and bond strength. The following equations are proposed to be used in the qualification tests using a BET as described before.

\[ \tau_{\text{max}} \geq 13 \cdot d_s^{-0.15} \]

\[ COV(\tau_{\text{max}}) \leq 7.5\% \]

\[ \tau_{0.1} \geq 8.5 \cdot d_s^{-0.15} \]

\[ COV(\tau_{0.1}) \leq 15\% \]

In the following Table, the given tests results are compared with the proposed criteria.

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<td>3.8</td>
<td>7.48</td>
<td>6.43</td>
<td>-</td>
<td>4.88</td>
<td>No</td>
<td></td>
<td></td>
</tr>
<tr>
<td>BE13</td>
<td>2</td>
<td>40</td>
<td>B500A</td>
<td>35.40</td>
<td>0.079</td>
<td>8.31</td>
<td>0.7</td>
<td>7.48</td>
<td>6.75</td>
<td>6.7</td>
<td>4.88</td>
<td>No</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

1) debonded length: 10 d, 2) predefined criteria for \( \tau \) -max and \( \tau \) -0.1

The results show that out of 26 test series, 11 were not qualified according to the criteria laid for the BET. All test series with a related rib area \( f_k \) smaller than 0.056 are identified and do not fulfil the bond stress criteria and/or the criteria for the maximum COV given in equation 6 and 7.

The reinforcing bars marked with an imprint are only qualified if the bar is tested with the imprint located outside of the bond length (see Table 2, series BE_stat_EP). If the imprint is located within the bond length (series BE_stat_EP_Pr), the scatter is larger than the provided criteria and therefore not qualified. In the test series, the COV-criterion is not fulfilled for \( \tau_{\text{max}} \) and \( \tau_{0.1} \). With test series BEV1 and BEV3, the COV-criterion for \( \tau_{\text{max}} \) is exceeded. Since all other criteria are clearly fulfilled,
this might indicate that the number of tests (4 (BEV1) and 2 (BEV4) should be increased. This is also the case for series BEV7, where the $\tau_{\text{max}}$-criterion is not fulfilled.

The results show that the proposed criteria are suitable to assess the bond behaviour of reinforcing steel bars with respect to the bond behaviour as an alternative for the related rib area on the basis of a Beam End Test.

7 Conclusions

To get more precise information about the bond slip behavior and the detailed knowledge of the Beam End Test there are still some parameters that should be investigated more in detail. In the common design formulae, the influence of the concrete compression strength is considered by a square root function. The tests show that this might not be completely realistic, especially for concrete strengths between 60 N/mm$^2$ and 120 N/mm$^2$. The influence of the related rib area $f_R$ turned out to be higher in the new tests than described in Ref.\textsuperscript{17}. This should be further investigated for different bar diameters and a larger variety of related rib areas.

The evaluation of newer tests showed, that the criteria according to Ref.\textsuperscript{17} have to be modified. Proposals for new criteria are given in this article.

As a conclusion, the Beam End Test is considering the above mentioned parameters, generally appropriate for testing static bond behavior of cast-in reinforcement bars. Moreover the testing procedure is connected with low effort and cost. The test specimen is relatively small and easy to handle. Furthermore, the test setup is well reproducible. Since this is important to ensure that the test is always conducted in the same way to get reproducible results the Beam End Test is generally suitable for the use in future standards.

8 Acknowledgement

The financial and technical support from the Institut für Stahlbetonbewehrung e. V. for the tests described in Ref.\textsuperscript{11} is greatly acknowledged. The authors also would like to acknowledge the collaboration of the technical committee "reinforcement" of the DAfStb (German Committee for Structural Concrete). Pursuant to a resolution passed by the German Federal Parliament (Deutscher Bundestag), the joint industrial research project (16992N/1)\textsuperscript{18} initiated by the Joint Committee for Cold Forming (Gemeinschaftsausschuss Kaltformgebung e.V.) was funded, via the German Federation of Industrial Cooperative Research Associations (Arbeitsgemeinschaft industrieller Forschungsvereinigungen = AiF), by the Federal Ministry for Economic Affairs and Energy (Bundesministerium für Wirtschaft und Energie) as part of the program supporting joint industrial collective research (Industrielle Gemeinschaftsforschung = IGF).

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A TEST METHOD FOR THE CHARACTERIZATION OF UNDERCUT ANCHORS

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ABSTRACT

The test program, requirements and criteria for undercut anchors is provided by European Assessment Document EAD 330232-00-0601 (former European Technical Approval Guideline ETAG 001 part 31). But a precise definition of undercut anchors is not given there. This can lead to confusion about the performance assessment and application. Some national specifications or building contractor require the use of undercut anchors for the fastening e.g. in nuclear power stations, in subways or in earthquake zones. Without a precise definition of undercut anchors it is difficult to distinguish the undercut anchor from the expansion anchor in practice.

“Testing splitting force of an anchor”2 seems to make the distinction between the traditional undercut and expansion anchor possible. But this test method is complex and always provides results with large deviation. A modern expansion anchor is developed in such a way that it generates minimal splitting forces. So the splitting force does not provide a clear differentiation between an undercut anchor and an expansion anchor.

In this study the different behavior of undercut and expansion anchor is discussed. The finite element simulations of pull out test of an expansion anchor were carried out for the investigation of the fastener’s function in cracked concrete. The sensitivity analyses evidence the significant installation condition for the load-bearing behavior of an expansion anchor. Thereby an approach is proposed to test the mechanical characteristic of an undercut anchor and to distinguish the two working principles of expansion und undercut anchors.

1 Introduction

Undercut anchors are post-installed fastening systems. They are applied in civil and structural engineering worldwide, e.g. in nuclear power stations, subways or earthquake zones. Some national and local specifications require the use of undercut anchor systems for such specific applications.

ETAG 001 Part 3 defines, that undercut anchors are characterised by mechanical interlock provided by undercut in the concrete1. ETAG 001 Part 2 provides the test program for expansion anchors and Part 3 for undercut anchors. They assess the robustness of both anchor systems by deviations in installation parameters. It is difficult to identify the mechanical character of undercut anchors and
expansion anchors by the provided test program only. For example, fischer anchor FAZ II is a high strength mechanical torque-controlled expansion anchor. By installing and loading it produces very low splitting forces like an undercut anchor. Its edge distance for splitting failure \(c_{es,sp}\) is 1.5 times of effective anchorage depth \(h_{ef}\). And this anchor is able to pass all the rigid suitability tests acc. to ETAG 001 part 3 provided for undercut anchors. Reversely, some undercut anchors were tested and could be assessed acc. to ETAG 001 part 2, which is actually provided for expansion anchors. This leads to confusion about the performance assessment and to uncertainty when confirming the undercut working principle.

The following questions arise:

- How can we distinguish between the character of undercut and expansion anchors?
- What is decisive for the distinction?

## Behavior of undercut and expansion anchors

Undercut anchors and metal expansion anchors exhibit similarities in load-bearing behavior. Also the design of anchorage for both anchors is identical. The main difference for both anchors is in the mechanical interlock.

### Undercut anchors

Undercut anchors develop a mechanical interlock between anchor and concrete. The EAD defines that the concrete is cut away rather than compressed\(^3\). The force is transferred from the fastening element (which fills out the room created by undercutting) to the concrete with the mechanical interlock. The mechanical interlock resistance ensures to bear the external tensile force in all conditions (Figure 1 a)). The friction between the expansion element and the concrete can be neglected because it does not influence the load bearing behavior strongly. So the condition of this working principle can be conservatively described in terms of forces resp. resistances as below:

\[
N_{ext} \leq R_{interlock}
\]  

where

\(N_{ext}\) = external force

\(R_{interlock}\) = resistance of mechanical interlock

### Mechanical expansion anchor

The major load-transfer mechanism employed by expansion anchors is friction. To avoid uncontrolled slip by loading, it is required that the frictional resistance between expansion element and concrete is larger than the external tensile force acting on the anchor in every application condition (Figure 1 b)).

\[
N_{ext} < R_{friction}
\]  

\(R_{friction}\) = frictional resistance
Feng Zhu and Andreas Bucher

Figure 1: Simplified anchor load transfer mechanisms: a) undercut anchor; b) expansion anchor

In fact the concrete is deformed plastically by high local compression due to the expansion forces applied by the expansion anchor. By installing an expansion anchor the ribs, knurling or other deformations on the expansion element is pressed in the concrete. This process creates a limited mechanical interlock. The modern expansion anchors transfer external tensile forces to the base material via friction and to a limited mechanical interlock in the region of the deformed concrete. The condition of this working principle of an expansion anchor can be described as below:

$$N_{ext} \leq R_{friction} + R_{interlock}$$

(3)

By anchor setting in an unfavorable condition the limited mechanical interlock can become weak. Figure 2 shows an example that the rib of an expansion anchor was damaged during anchor setting in the cylindrical hole drilled with a minimum diameter of the drill bit in high strength concrete. In this case the friction is the decisive factor for ensuring the function of the expansion anchor in cracked concrete and then the anchor’s tensile load-bearing performance depends on friction only.

Figure 2: Friction is the major load-transfer mechanism employed by expansion anchors with weak interlock.
The friction resistance of expansion anchors is theoretically dependent on the expansion force and friction coefficient:

\[ R_{\text{friction}} = \mu \cdot P \]  

\( \mu \) = friction coefficient between fastening element and concrete hole

\( P \) = pressure due to expansion force

Comparing equations (1) and (3) evidences that the friction resistance distinguishes between undercut and expansion anchor. One could minimize the influence of friction in the test program acc. to the ETAG 001 part 3\(^1\), so that the test results indicate only the robust mechanical characteristic of interlock for undercuts.

Two approaches are possible to minimize the friction resistance by testing undercut anchor:

1) **Reducing the friction coefficient between the fastening element and concrete:** Figure 3a) shows the load-displacement-curves of the suitability tests for an expansion anchor without cleaning the drill hole gained in cracked concrete C50/60 with a crack width \( \Delta w = 0.5 \text{ mm} \). The uncontrolled slip occurs due to lack of friction and mechanical interlock. Under the same test condition undercut anchors have not shown uncontrolled slip (Figure 3). The undercut anchor behavior is not sensitive to friction coefficient (which depends on the cleaning process) due to the robust mechanical interlock.

2) **Reducing the pre-stress before the crack opens:** After the installation of the anchor, the torque moment is reduced completely before the crack opens. Due to reducing the torque moment no pre-stress exists. When the crack opens the expansion anchor does not permit follow-up expansion. The capacity of the non-pre-stressed expansion anchor is thus influenced by cracks to a much greater degree than the capacity of an undercut anchor. Figure 4 shows two load-displacement curves gained with an expansion anchor in cracked concrete: One with uncontrolled slip due to insufficient pre-stress (red curve) and one with the provided pre-stress by the prescribed torque moment (blue curve). This case may happen in practice if the pre-stress would be lost before the crack opens.

![](image1.png)

\( \text{a) an expansion anchor} \quad \text{b) an undercut anchor} \)

**Figure 3:** Load-displacement-curves of a reliability test without hole cleaning gained in cracked concrete C50/60 and crack width \( \Delta w = 0.5 \text{ mm} \) for a) an expansion anchor and b) an undercut anchor.
Figure 4: Load-displacement behavior of an expansion anchor installed in cracked concrete. The blue curve represents the behavior of an expansion anchor with the determined pre-stress; the red curve shows the uncontrolled slip by the expansion anchor without pre-stress before the crack opening.

3 Numerical Investigations

In order to investigate the influence factors on friction in test conditions for an anchor the numerical sensitivity analysis was performed with the program Ansys®.

An FE-model was developed to simulate the expansion process of an expansion anchor (size M24) and pull out test of the anchor in cracked concrete (Figure 5a)). The FE-model was calibrated with the test results. In Figure 5b) the successful load transfer from expansion element to the concrete is shown. To investigate the friction resistance without a mechanical interlock the concrete is assumed as linear elastic in this model.

Figure 5: a) FE-model; b) Successful load transfer from fastening element to the concrete

The FE-model is able to indicate the load-displacement behavior of an expansion anchor under different test conditions. In Figure 6 the simulation results in terms of the load-displacement
behavior of an expansion anchor are shown reflecting the influence of the installation process (pre-stress) and the crack opening. Thereby the same expansion anchor was simulated under different installation conditions. By different input parameters different conditions were achieved. The response is the load-displacement curve.

To study the sensitivity of the friction condition of an expansion anchor, 60 combinations of installation conditions were stochastically simulated using the same expansion anchor geometry. All parameters, e.g. friction coefficient, drill hole diameter, crack width and pre-stress of anchor, were used as input parameters (Table 1). As a matter of fact, a Latin Hypercube Sampling of 60 samples of installation conditions was sufficient to achieve a coefficient of prognosis of 83% for the most important response values. The two parameters, drill hole diameter and pre-stress before crack opening (Table 1), have the highest impact on the load-displacement behavior: The drill hole diameter has the significant influence with 48% and the pre-stress with 28% on the load-displacement behaviors. Figure 7 shows the 3D response surface of maximum load depending on the parameters of drill hole diameter and pre-stress of the anchor.

Table 1: Range of parameters and Coefficients of Prognosis

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Input Range</th>
<th>Coefficients of Prognosis</th>
</tr>
</thead>
<tbody>
<tr>
<td>crack width</td>
<td>0.3 – 1.0 mm</td>
<td>9%</td>
</tr>
<tr>
<td>friction coefficient</td>
<td>0.3 – 0.55</td>
<td>13%</td>
</tr>
<tr>
<td>pre-stress</td>
<td>5 – 41 kN</td>
<td>28%</td>
</tr>
<tr>
<td>drill hole diameter</td>
<td>24.1 – 24.5 mm</td>
<td>48%</td>
</tr>
</tbody>
</table>
Figure 7: 3D response surface of maximum load dependent on the parameter of drill hole diameter and pre-stress

4 Result

Since an elastic concrete material is assumed in the FE-model, the simulation results represent only the friction load-transfer behavior. The interlock based on a plastic deforming of concrete has no impact on the load bearing behavior of the anchor in the FE simulation. The simulations show that the parameters “drill hole diameter” and the “pre-stress” of an anchor together have significant influence on the friction resistance of an expansion anchor (76 %). Cleaning the drill hole (friction coefficient) has 13 % influences. In ETAG 001 Part 3 the sensitivity tests are specified for the drill hole diameter\(^1\). In these tests an additional test condition that reduces the torque moment (\(T_{\text{inst}} \rightarrow 0\)) before the crack opening can minimize the share of friction of an anchor effectively. Therefore, these tests are suitable to determine the mechanical interlock characteristic and to distinguish the two working principles of undercut and expansion anchors. On the basis of this test method the main working principle of the product can be determined as interlock, if the requirements on the load-displacement behavior given in ETAG 001 Part 1, 6.1.1.1\(^5\) are kept without pre-stress of the anchor system in the tension direction before crack opening.

5 Conclusion and Outlook

For practical application it is often required to distinguish an undercut anchor from an expansion anchor. By testing mechanical interlock characteristics it is possible to distinguish the two working principles. The stochastic FE-simulations show that the main influencing parameters on friction are the drill hole diameter and the pre-stress of an anchor. In order to clearly determine the interlock characteristics one can carry out the sensitivity tests for an undercut anchor by minimizing the friction condition with reducing the pre-stress of the anchor before crack opening.

To proof the suitability of the above described test method in general, a workbench of investigation for different anchor types and anchor diameters should be performed.
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COMPOSITE STRUCTURES
CHARACTERIZATION OF LOAD-TRANSFER MECHANISMS AT THE STEEL-CONCRETE INTERFACE IN REINFORCED CONCRETE ELEMENTS STRENGTHENED WITH STEEL PROFILES

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ABSTRACT

The European research project Smartcoco (Smart Composite Components) aims at preparing a design methodology to be used for all situations in which steel profiles are used to locally reinforce concrete structures. Such reinforcing may cover, for instance, practical situations such as composite column sections with more than one fully encased steel profile, columns of RC buildings locally reinforced by discontinuous steel sections, composite elements used to transfer forces in zones of discontinuity in RC walls, or coupling beams above and below openings in concrete cores of buildings. The first requirement in the development of such a methodology is the proper characterization of the bond (or longitudinal shear transfer) properties at the interface between concrete and steel profiles. To this purpose, a set of push-in tests was carried out at the Material & Structures Laboratory of the University of Liège. The aim of these tests was to study the transfer of compression/tension forces from the steel profile to the concrete wall by longitudinal shearing of the interface without creating local disturbances, like transverse cracking or splitting of concrete around the steel profile. 10 tests specimens were investigated, with flexible (shear studs) and/or rigid (steel plates) shear connectors, for different orientations of the profile. The purpose of the paper is to present the experimental results and the use of these results to calibrate practical design formulae.

1 Introduction

Although a number of researches have focused on various aspects of composite structures, no design guidance exists for typical situations that are more and more common in the current construction practice, such as concrete columns reinforced by several steel sections, concrete columns reinforced by steel sections over only one structural level or composite connections of flat slabs to columns or walls. Such configurations can be qualified as “hybrid” structural situations since they are neither reinforced concrete structures in the sense of Eurocode 2¹ or ACI 318², nor composite steel-concrete structures in the sense of Eurocode 4³ or AISC 2010⁴.
The final goal of the European project Smartcoco (Smart Composite Components, funded by the Research Fund for Coal and Steel of the European Commission) is the production of a design guide to be used for all situations in which steel profiles are used to reinforce locally what for else remains a reinforced concrete structure. The intention is to first present a tentative unified approach for such situations and then to test the approach for a set of different structural situations. The generic approach is tentatively based on “strut and tie” approaches, but such an approach, although well settled in reinforced concrete structures, is still to be developed and calibrated in many situations for hybrid structures in which one or several steel profiles are used as local reinforcements or as connections either between RC elements or between RC and steel elements.

Gaps in knowledge are common to all types of hybrid elements, as they are mostly related to the problem of force transmission between concrete and embedded steel profiles, a situation in which it is not known how to combine the resistances provided by bond, by stud connectors and/or by plate bearings, and how to reinforce the concrete in the transition zones between classical reinforced concrete and composite in order to avoid damaging effects due to curved stress flows. As detailing and design of the steel-concrete interface are to a large extent similar in all hybrid elements, the intention is to come up with design rules for generic interface problems, as it should be expressed in a design code, and to verify their practicability and efficiency by applying them to the specific structural members that are representative of common situations encountered in actual practice.

As a matter of examples, the final methodologies should be able to handle practical situations such as the following, illustrated in the final report Smartcoco:\textsuperscript{5}:

- Composite column sections with more than one fully encased steel profile;
- Columns of RC buildings reinforced by interrupted steel sections;
- Composite elements used to transfer forces in zones of discontinuity in RC walls;
- Coupling beams above and below openings in concrete cores of buildings;
- Flat slabs.

As a necessary step in the global design procedure, a specific intention is required regarding the shear transfer mechanisms at the interface between the steel profiles used as longitudinal reinforcement and the surrounding concrete, similarly to the classical bond characterization between longitudinal reinforcing bars and concrete.

2 Description of the test specimens

2.1 Investigated parameters

The main purpose of the investigations summarized in the present contribution was to compare the efficiency of different solutions to transfer compression/tension forces from a steel profile to the
surrounding concrete without creating local disturbances, like transverse cracking or splitting of concrete around the steel profile. In total, 10 tests have been carried out, corresponding to 6 main configurations and including flexible and stiff connectors and different orientation of the profile with respect to the smallest dimension of the global composite specimen. The different configurations are summarized as follows and illustrated in Figures 1 to 4. It is also relevant to note that, for all specimens, no end-plate is welded to the profile to ensure a full load transfer via longitudinal shear.

- **Configuration A**: steel profile with strong axis perpendicular to the longer wall face; connections with 6 flexible connectors (d = 16 mm / h = 65 mm / S235) on the total length of the steel encased profile; transverse links at each connector - 1 test;

- **Configuration B**: steel profile strong axis parallel to the longer wall face; connections with 4 stiff connectors (“transverse stiffener” type); transverse links as from strut and ties design - 3 tests with different reinforcement configuration (B1, B2 and B3);

- **Configuration C**: steel profile weak axis perpendicular to the longer wall face; connections with 6 flexible connectors on the total length of the steel encased profile; transverse links at each connector: 1 test;

- **Configuration D**: steel profile weak axis perpendicular to the longer wall face; connections with 4 stiff connectors (“transverse stiffener” type); transverse links as from ‘strut and tie’ design - 2 tests with different reinforcement configuration (D1, D2);

- **Configuration E**: steel profile weak axis perpendicular to the longer wall face; connections with 2 flexible and 2 stiff connectors on the total length of the steel encased profile; transverse links at each connector - 1 test;

- **Configuration F/G**: steel profile strong axis perpendicular to the longer wall face; respectively with rust and paint on profile (no mechanical connectors) – 2 tests.

![Figure 1: Push-in tests – configurations A and C](image-url)
Figure 2: Push-in tests – configuration B

Figure 3: Push-in tests – configuration D

Figure 4: Push-in tests – configurations F/G
2.2 Design procedure

For configurations A and C, the design resistance of the shear-studs connection is estimated according to Eurocode 4\(^6\). Vertical reinforcement scheme of the concrete elements is based on technological requirements from Eurocode 2. Additional horizontal reinforcements are then mandatory to anchor the compression struts associated to the transfer of forces from the profile to the concrete through the shear studs. For configuration B and D, a strut and tie mechanism is also considered to control the load transfer from the plates, while the resistance of the plate connectors themselves is determined assuming a local yield mechanism. The detailed calculation and related strut and tie mechanisms assuming compression struts developing with an angle of 45° can be found in the detailed publications related to this project\(^7,8\).

Figure 5 (left) illustrates the steel configuration of the specimens before concreting while Figure 5 (right) shows the test specimens in final configuration. The measured characteristic strength of concrete is equal to \(f_{ck} = 71\) MPa. The concrete appears of a much better quality than assumed for the design of the test specimens, resulting in a failure by local plastic crushing of the protruding part of the embedded profile, and not by concrete-related failure mechanisms. In order to overcome this problem, it was required that some test specimens be cut to one half of their initial length, resulting in tested concrete blocks with a height of 500 mm instead of 1000 mm, as shown on Figure 6 for one of the cut specimens installed in the test setup.

Figure 5: Test specimens. Steel part before concreting (left) – Specimens before testing (right)

Figure 6: push-in tests – test specimen within the test rig
3 Test results and main observations

The full set of results available in detailed test reports. As a matter of illustration, Figure 7 (left) shows a comparison of push out curves for configurations F and G (specimens without mechanical connectors and with different state of surface of the profile, i.e. rusted and painted) corresponding to a typical cohesive behavior, while Figure 7 (right) compared the curves obtained for the tests with plate connectors. Typical failure modes are illustrated in Figure 8, mainly showing a set of vertical or sub-vertical set of cracks associated to the failure of the interface and the development of a strut mechanism.

![Illustrative push-in curves. Comparison rusted vs. painted (left) – Comparison of different configurations with plated connectors (right)](image)

![Failure mode of specimens B1 (left) and F (right)](image)

Table 1 summarizes:

- The characteristics of the specimens for what concerns resistance to longitudinal shear at steel profile-concrete interface;
• The estimation $V_{RD}$ of the strength based on Eurocode 4 design expression or on design formulas specifically derived within the project Smartcoco, in case nothing had been available in Eurocode 4;

• An experimental value $V_{R,Exp}$, which is the value of $V_{Exp}$ at 0.5 mm slip. This value has been chosen as being the experimental estimate of the yield resistance.

Table 1: Synthesis of calculated and experimental resistances

<table>
<thead>
<tr>
<th>Spec.</th>
<th>Connectors</th>
<th>Transverse reinforcement</th>
<th>Web orientation</th>
<th>$V_{RD}$ Bond (kN)</th>
<th>$V_{RD}$ Conn. (kN)</th>
<th>$V_{RD}$ Total (kN)</th>
<th>$V_{R, Exp}$ (kN)</th>
<th>$V_{R, Exp}/V_{RD, tot}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>6 studs on flanges</td>
<td>1 link/stud</td>
<td>//</td>
<td>232</td>
<td>434</td>
<td>666</td>
<td>1050</td>
<td>1.57</td>
</tr>
<tr>
<td>B1</td>
<td>4 plates</td>
<td>No</td>
<td>//</td>
<td>0</td>
<td>678</td>
<td>678</td>
<td>600</td>
<td>0.88</td>
</tr>
<tr>
<td>B2</td>
<td>4 plates</td>
<td>6 hoops</td>
<td>//</td>
<td>232</td>
<td>678</td>
<td>910</td>
<td>920</td>
<td>1.01</td>
</tr>
<tr>
<td>B3</td>
<td>4 plates</td>
<td>6 links</td>
<td>//</td>
<td>232</td>
<td>678</td>
<td>910</td>
<td>740</td>
<td>0.81</td>
</tr>
<tr>
<td>C</td>
<td>6 studs on flanges</td>
<td>1 link/stud</td>
<td>T</td>
<td>222</td>
<td>434</td>
<td>656</td>
<td>910</td>
<td>1.38</td>
</tr>
<tr>
<td>D1</td>
<td>4 plates</td>
<td>No</td>
<td>T</td>
<td>(222)</td>
<td>678</td>
<td>900</td>
<td>1000</td>
<td>1.11</td>
</tr>
<tr>
<td>D2</td>
<td>4 plates</td>
<td>6 links + 6 hoops</td>
<td>T</td>
<td>222</td>
<td>678</td>
<td>900</td>
<td>1000</td>
<td>1.11</td>
</tr>
<tr>
<td>E</td>
<td>2 plates + 2 studs on flanges</td>
<td>6 links + 6 hoops</td>
<td>T</td>
<td>222</td>
<td>144 + 339 = 483</td>
<td>705</td>
<td>700</td>
<td>0.99</td>
</tr>
<tr>
<td>F</td>
<td>0 (rusted)</td>
<td>No</td>
<td>//</td>
<td>514</td>
<td>514</td>
<td>514</td>
<td>1480</td>
<td>2.88</td>
</tr>
<tr>
<td>G</td>
<td>0 (painted)</td>
<td>No</td>
<td>//</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>580</td>
<td>$\infty$</td>
</tr>
</tbody>
</table>

4 Conclusions and design recommendations

Based on a detailed analysis of the outcomes illustrated here above, the following conclusions have been reached:

• Bond resistance for encased profiles defined in Eurocode 4 is in all case safe-sided, but the bond resistance $\tau_{RD}$ reached in the tests is almost 3 times greater than the Eurocode 4 design value and still 2 times greater even if scatter would be considered.

• The bond resistance of the painted specimen is greater than Eurocode 4 design value for rusted specimens (0.3 N/mm²) and much greater than the “0” N/mm² indicated for painted specimens.

• Connections using plate connectors can be calculated using the method proposed in the Smartcoco project. However, for H profiles oriented with the web parallel to the wall face,
transverse reinforcement by hoops placed around the profiles shall be provided to equilibrate the compression-strut forces; otherwise a brittle failure is observed.

- For H profiles oriented with the web perpendicular to the wall face, transverse reinforcements are not required for internal profiles. These are nevertheless required for profiles situated in the edge regions of the wall.

- Contribution to shear resistance provided by bond and all type of connectors can be added to evaluate the total available longitudinal shear resistance.

Beyond the promising qualitative observations reached so far, a wider experimental basis would now be required to provide extensive statistical data, indication on the scattering and suggestion of partial safety factors.

5 Acknowledgement

The authors acknowledge the support received from the Research Fund for Coal and Steel of the European Commission (RFCS) – Research grant agreement RFSR-CT-2012-0003 Smartcoco.

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2. ACI318-08 (2008). Building code requirements for structural concrete and Commentary, American Concrete Institute


EXPERIMENTAL INVESTIGATION ON PRECAST COLUMN
CONNECTIONS UNDER CYCLIC LOADING

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ABSTRACT

Precast structures offer several advantages during both production and installation compared with cast-in-situ structures. However, the connections between precast elements play a critical role in the overall performance of such structures. This is particularly relevant in case of seismic applications, where structural continuity is needed. In fact, the response of connections under horizontal cyclic loading is of primary importance in order to guarantee both resistance and deformation capacity of the structure, especially if column-to-foundation joints are considered. A series of full-scale precast column-foundation sub-assemblies have been tested aiming to investigate the performance of bolted joints made with column shoes and anchor bolts. Such performance was compared to that of two cast-in-situ columns in terms of ductility, energy dissipation, stiffness and strength degradation. The results showed that the seismic behavior of properly designed bolted joints is equivalent to that of cast-in-situ connections, which allows to consider those as energy dissipative according to current design Codes.

1 Introduction

The connections between precast elements affect considerably the overall resistance and deformation capacity of the structure, especially if seismic applications are considered. In fact, precast connections represent a critical link where structural continuity is needed. Early precast constructions had inadequate detailing and lacked of continuity or redundancy in the structure, resulting in poor seismic performance. Furthermore, there were no design guidelines for precast concrete structures used in seismic areas.

For these reasons, precast has seen limited use in earthquake-prone zones. So far, engineers have favored cast-in-situ solutions or used alternatives such as protruding bars or hybrid connections out of habit. However, there is not always clear evidence of their seismic behavior and usage risk evaluations are lacking. In addition, precast structures offer several advantages during both production and installation compared with traditional solutions, such as better material and product quality control, improved erection speed and cost savings.

In the last two decades, numerous studies have been performed regarding the cyclic behavior of precast joints. At the same time, remarkable progress has been achieved in the development of modern Codes\textsuperscript{1,2,3,4,5}. Specific requirements for both strength and ductility of the connections have
been imposed so that structures can withstand seismic load reversals without a substantial reduction in global resistance.

With regard to such background, this paper refers to a wide experimental campaign carried out by the cooperation between Peikko Group and Politecnico di Milano (Technical University of Milano), which investigated the performance of bolted column-to-foundation connections made with column shoes and anchor bolts\textsuperscript{6,7}. The aim of the research was to develop a precast connection that emulates the monolithic joints with the same performance in terms of ductility, energy dissipation capacity, stiffness and strength degradation, thus combining compliance with the Codes with the advantages of precast structures.

2 Precast bolted connections

Figure 1a shows a Peikko column-to-foundation connection. HPKM\textsuperscript{®} Column Shoes\textsuperscript{8} are assembled from base and lateral steel plates and anchoring rebars, which are cast at the base of the precast element. Weldings between these components have a nominal strength at least twice that of the anchor bolts. This guarantees the elastic response of the welds. HPM\textsuperscript{®} Anchor Bolts\textsuperscript{9}, instead, are ribbed steel bars, which are partly casted into the foundation. The external threaded part allows the base plate to be tightened using two washers and two hexagonal nuts.

The open joint between the column and foundation, including column shoes pockets, is then filled on site with non-shrink cementitious grout. The grout has a design compressive strength at least one class higher than the highest grade of concrete used in the connected elements, so that brittle concrete failures are avoided in the joint.

Figure 1: (a) Precast bolted column-to-foundation connection\textsuperscript{6,7}; (b) Chain analogy of the design alternatives.
2.1 Design alternatives for the connection system

Generally, the Codes allow two different approaches to design precast column-to-foundation connections (Figure 1b). Firstly, connections can be overdesigned so that the critical region moves to the connected column. The connection remains almost elastic with limited displacements or local deformations, representing a “strong” link in the hierarchy of resistances. However, the area of the column above the joint is over-strengthened due to the overlapping of column shoe rebars and column reinforcement. Therefore, a design of the connection that is based on the resistant moment of the column that it supports is not convenient, since it requires relatively large column cross-sections to fit the necessary anchoring bolts and might result in dense reinforcement in the joint.

The second alternative is represented by energy dissipating connections, which are located in the critical region but also comply with the prescribed local ductility criteria. In this case, the plastic hinging of the column and/or the buckling of the rebars are avoided while the possible damage is limited to the base of the column at the interface with the foundation, where the anchor bolts represent the “weak” element and act as ductile connectors. In contrast to overdesigned connections, the resistance of energy dissipating connections is dependent on the acting moments as for cast-in-situ joints. Since the joint dissipates energy itself, it can be designed to match the capacity of the column, while respecting the capacity design in the overall structure. Under specific conditions, this leads to a smaller and adequately reinforced column cross-section.

2.2 The emulative precast connection

To be considered “ductile”, a connection must show experimentally a stable cyclic behavior and an energy-dissipative capacity at least equal to that of a monolithic connection that has the same resistance and conforms to the local ductility provisions of the Code. Special detailing was then introduced in Peikko’s standard column connection for this purpose (Figure 2). The effectiveness of the new features is evaluated basing on the comparison between earlier experimental results, where such improvements were not yet included.

HPM®-EQ Anchor Bolts were specifically developed and produced with B500C, which is the highest ductility steel material. The embedded thread is now debonded by a heat shrink tube so that the anchor bolt is able to deform freely and the deformation capacity of the steel is not reduced. Loads are then transferred through the ribs and the headed stud as in standard anchor bolts.

The tightening of the joint under cyclic loading is secured by high strength and anti-lock washers as well as by a type of pre-tensioning of the anchor bolts, which is induced by an additional rotation of the upper nut after snug tightening. An epoxy resin is injected around the anchor bolt inside the oversized hole of the base plate in order to compensate for the tolerance needed during installation. This helps to significantly reduce the pinching effect, which results in an increased amplitude of hysteresis cycles as experimentally verified.

A high strength fiber-reinforced mortar is used as joint grouting to avoid the spalling of the unconfined compressed collar of mortar around the column base. Moreover, the surfaces at the base of the column and on the top of the foundation are indented, so that compressed struts can develop between the upper and lower indentations. The shear resistance of the joint relies on both the friction and the mechanical interlocking of the surfaces. Shear is mainly resisted by this mechanism, while
anchor bolts are subjected almost exclusively to tension and compression. Finally, additional stirrups around the column shoes limit their mutual displacements and rotations, thus reducing the cracking of the joint.

Figure 2: Emulative column-to-foundation connection.

3 Experimental investigation

In order to assess the performance of such an innovative connection for seismic applications, several full-scale sub-assemblies consisting of a 2.15 m high column and a rigid foundation element have been tested at *Politecnico di Milano* (Figure 3a).

Different layouts of the connection were investigated by changing the number and size of the anchor bolts and by varying the column’s cross-sectional dimensions. Some configurations also underwent three identical tests to assess the replicability of the results. For the sake of brevity, the results presented in the following refer to the connection arrangements shown in Figure 3b.

Since the main aim of the research activities was to compare the cyclic performance of the emulative connection to that of a cast-in-situ joint, two monolithic columns, which complied with reinforcement detailing for high ductility class as required by CEN\(^{1,15}\), were therefore tested. Such columns were designed to be equivalent to the precast specimens in terms of resistance as indicated in Figure 3b.
3.1 Test setup
Quasi-static oligo-cycles imposed-displacement tests were performed. Both the typologies of the specimens were tested by applying the same drift pattern with three cycles of equal displacement for each increasing drift level (0.5%, 1%, 2%...) until failure. The failure criteria were anchor bolt failure or a loss of horizontal resistance greater than 20% from the peak value. Columns were also vertically loaded with a constant axial ratio of about 10%.

3.2 Test results
The precast specimens all showed a localized damage in connection to the grouting, which presented an extensive crack pattern at the end of the test (Figure 4a). Spalling of the mortar was avoided thanks to the steel fibers, which kept the mortar in place around the cracks. It is worth noting that little or no damage was observed for drifts of up to 1%, which is beyond the limit for inter-story drift imposed by the Code. Even after a moderate earthquake the column would remain almost undamaged and any possible repair intervention would affect the grouting only.

In order to investigate the ultimate capacity of the connection, the tests continued until failure, which always occurred for drifts greater than 5%. This highlights the great deformation capacity of the connection, which relies on the anchor bolts. Anchor bolts failed generally below the lower nut or the foundation level since the concentration of the stresses was maximum at the interface between column and foundation as expected. Moreover, the thread of the anchor bolts emerging from the foundation was generally damaged, which is possibly due to tensile and compressive cyclic loading (Figure 4a).

Conversely, cast-in-situ specimens suffered generalized damage with evident spalling at the base of the column and cracks on the foundation surface (Figure 4b). This would lead to higher repair costs. Furthermore, the longitudinal reinforcement buckled and one of the rebars failed in CIP1 (Figure 4b).
This indicates that brittle failure could easily occur, especially in absence of proper detailing such as adequate confinement of the critical zone.

Figure 4: (a) PC1 specimen and failed anchor bolts with damaged thread; (b) CIP1 specimen with buckling and failure of longitudinal rebars.

All the precast specimens achieved a displacement ductility\(^{16}\) of at least 4, showing great post-elastic deformation capacity (Table 1). In particular, Figure 5a shows the comparison between the force-displacement curves of PC1 and CIP1. It can be noticed that the displacement at failure of the precast specimen is greater than that of the correspondent cast-in-situ one. Moreover, the strength degradation of the precast specimen is extremely limited, fulfilling the threshold (< 20%) recommended by ICBO\(^{17}\), while the cast-in-situ column suffered an abrupt loss of resistance after 4% drift due to rebar buckling and spalling.

Table 1: Ductility values for positive and negative displacements.

<table>
<thead>
<tr>
<th>Test</th>
<th>PC1</th>
<th>PC2</th>
<th>PC3</th>
<th>CIP1</th>
<th>CIP2</th>
</tr>
</thead>
<tbody>
<tr>
<td>+Δ</td>
<td>Δ(_y)</td>
<td>0.8</td>
<td>1.3</td>
<td>1.5</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>μ(_d)</td>
<td>8.8</td>
<td>6.2</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>−Δ</td>
<td>Δ(_y)</td>
<td>0.8</td>
<td>0.9</td>
<td>1.7</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>μ(_d)</td>
<td>8.8</td>
<td>6.3</td>
<td>3.5</td>
<td>3.3</td>
</tr>
</tbody>
</table>

The comparison between the backbone curves of all the specimens confirms that the tested precast and cast-in-situ columns are similar in terms of resistance according to the design (Figure 5b). PC2 specimen also showed a greater deformation capacity than PC1, thanks to the presence of more anchor bolts of a smaller diameter. The CIP2 specimen was the only one that did not reach failure, even at drifts of more than 9%, and it showed better performance than the equivalent precast PC3 specimen. This was due to the continuous reinforcement between the column and foundation in CIP2, which was designed according to the requirements for special moment frames\(^{18}\).

Precast and cast-in-situ specimens showed also similar energy dissipation, the precast specimens showing even more stable hysteresis cycles. The equivalent damping factor has been evaluated by summing the elastic (2%) and the hysteretic components. The result was in general greater than 37.5%, thus fulfilling the requirements in ICBO\(^{17}\) for ductile connectors (Figure 6a). Finally, Figure 6b shows that bolted connections can provide also enough stiffness\(^{16}\) as the monolithic joints. It can
also be noticed that the decay of initial stiffness is gradual, without any sudden and undesirable stiffness loss.

![Figure 5](image)

Figure 5: (a) Comparison of the hysteresis curve of PC1 and CIP1 specimens; (b) Comparison of the backbone curves.

![Figure 6](image)

Figure 6: Comparison of the damping factor (a) and the initial stiffness (b).

4 Conclusion

The presented research program addressed the challenges of seismic design of bolted joints and the requirements imposed by the current Codes for precast structures. The response to horizontal loading of column connections made with column shoes and anchor bolts has been investigated through a series of quasi-static cyclic tests. The experimental results showed that such connections can resist seismic loads with a satisfactory ductility and stiffness.

In particular, the innovative connection with partly debonded anchor bolts and improved construction detailing showed a stable and dissipative cyclic behavior. The failure of precast specimens always occurred at drift greater than 6%, with a displacement ductility equal to or greater than 4 and a limited strength degradation.

The comparison with two cast-in-situ columns proved that bolted connections, if properly designed, are equivalent to monolithic joints, thus avoiding to be overdesigned according to the Codes. This
can lead to substantial savings, both in concrete usage and in the building process, making it a fast precast system for installation on the construction site. In general, such results are promising for the use of bolted connections as a safe, reliable and convenient solution for precast concrete structures in seismic areas.

5 Acknowledgement

We thankfully acknowledge Professor Lorenzo Jurina and Edoardo Radaelli (Department of Architecture, Built Environment and Construction Engineering, Politecnico di Milano) for the active cooperation in the research program and the technicians Franco Gaggero, Marco Antico and Andrea De Steffani (Testing Materials Structures and Constructions Laboratory) for the assistance given during the tests.

References:


SHEAR CONNECTIONS IN COMPOSITE GIRDERS WITH CORRUGATED WEB

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ABSTRACT

Composite steel-concrete girders exhibit many advantages in terms of structural capacity, economy and easy and quick construction, they represent a valuable alternative to prestressed concrete bridges. This paper presents an innovative composite section, which consists of a steel girder with a trapezoidally corrugated web, of which the upper flange has been removed and horizontally lying studs have been welded to the web. The connection steel to concrete is performed by the horizontally lying studs and the partial embedment of the steel web into the concrete. The section design allows a reduction of the welding and of the steel consumption. In previous research from the authors the longitudinal shear resistance of the connection with one layer of horizontal studs was experimentally and numerically analyzed, obtaining a design equation to characterize the longitudinal shear strength.

In addition, this paper reports on the numerical study of a second variant of this girder, which introduces a supplementary layer of studs connecting the steel girder with a prefabricated concrete slab. So, together with the constructive advantages, an enhancement of the longitudinal shear capacity is achieved.

1 Introduction

The composite girder with a corrugated steel web is presented here as a competitive alternative against other composite or concrete girder for bridges in the range of more than 30 meters span. The girder section consists of a corrugated steel girder without the upper flange but with horizontally lying studs welded to the steel web, Figure 1. The studs are distributed into two rows, the lower one is embedded into a precast concrete slab, and the upper one into in-situ concrete.
The corrugated steel girder is frequent in composite bridges in Japan and it can also be found in some bridges in Europe. The prefabricated concrete slab has been also used in some existing bridges, for example, in the Straßenbrücken in Münsingen, where the steel web was flat and not corrugated. In the Figure 2 the composite girder with the prefabricated concrete can be observed. The Figure 3 shows the girder already placed in the final location prepared for the pouring of the in-situ concrete.

The composite girders with corrugated steel sheet have been studied at the University of Stuttgart under different types of loadings\textsuperscript{1-9}: longitudinal and transversal shear, static and dynamic loading. This paper describes first the analysis of the longitudinal shear strength for the connection with one layer of studs, that is the starting point of the second part of the research here presented, that reports the numerical analysis of the connection with two layers.

2 Longitudinal shear strength of the connection with 1 layer of studs

2.1 Experiments

In the framework of the FOSTA P645\textsuperscript{1} an experimental program of push-out tests (Figure 4) was carried out on specimens of composite girders with corrugated steel web. Some of the specimens were tested without studs, so the connection was only achieved by means of the embedment of the
corrugated steel web into the concrete. The aim was to know the effect of the studs on the shear capacity of the connection. Together with this effect, the influence of different arrangements of the studs was studied.

The curves of the Figure 5 represent the relative shear load –slippage in the connection extracted from the push-out tests. The higher capacity in the specimens with headed studs welded to the steel web compared with the specimens without studs (blue curves) was observed, as well as the better behavior when the number of studs is higher.

2.2 Numerical study and formulation

The connection involves multiple parameters that are expensive and difficult to cover only with experimental tests. Therefore, in order to consider more variables and assess their influence on the shear capacity of the connection, a FE modelling using the software MASA\textsuperscript{7,8} was developed.

Figure 4: Test setup push-out test

Figure 5: Relative shear load –slippage (mm)

curve from push-out tests

Figure 6: Main tensile strains of concrete chord after exceeding ultimate load; a) view from the outside, b) view from the inside without corrugated steel web
Figure 6 shows the main stresses after exceeding the ultimate load. The biggest deformations shown in red are in agreement with the cracks and concrete cone failure observed in the experiments, they grow from the headed studs in direction to the concrete surface.

Through the tests, the numerous finite-element calculations, as well as a statistical evaluation according to EN 1990, an equation for the longitudinal shear strength $P_{tL}$ for the connection was derived as a function of the most influential variables. The capacity of the concrete $f_{ck}$ ($\leq 50$MPa) together with the characteristic friction steel-concrete $\mu_k$ (at minimum 0.05) and the height of the steel web embedded into the concrete $t_e$ (between 175 and 275mm), have the highest effect on the longitudinal shear strength of the connection. Figure 7 shows other geometrical parameters.

$$ P_{tL} = \frac{V}{\rho \gamma_v} \cdot k_b \cdot k_s \cdot k_\alpha \cdot k_{a1} \cdot (f_{ck})^{0.6} \cdot (\mu_k)^{0.3} \cdot (t_e)^{0.35} \cdot (t_w)^{0.2} \cdot (a_{sw})^{0.05} \cdot (1 + a_s a)^{0.05} \quad [KN/m] \quad (1) $$

- $k_b$ coefficient for the lateral boundary constraints of the concrete slab;
  - with headed studs: $k_b = 1.57 \cdot (n)^{-0.28} > 0.5$ with $n = n^o$ periods;
  - for no constraint $k_b = 0.5$
  - for total constraint of the displacements $k_b = 1.5$
- $k_s$ coefficient for the studs arrangement, Figure 8;
- $k_\alpha$ coefficient for the angle of the corrugation $\alpha$:
  - $17^\circ < \alpha \leq 30^\circ$: $k_\alpha = 0.02 \cdot (\alpha) + 0.4$
  - $30^\circ < \alpha \leq 45^\circ$: $k_\alpha = 1$
  - $45^\circ < \alpha \leq 90^\circ$: $k_\alpha = -0.007 \cdot (\alpha) + 1.31$
- $k_{a1}$ coefficient for the length of the corrugation, with
  - $140 < a_1 < 430$ mm; $a_1 = a_2$
  - $a_1 = 140$ mm; $k_{a1} = 1.0$
  - $a_1 \geq 225$ mm; $k_{a1} = 0.8$
  - interpolation for intermediate values, $a_1 = 1.0$ for free boundary conditions
- $V$ coefficient for adaptation experiments and calculated values, $V = 11500$;
- $P$ projected length of the corrugation mm;
- $\gamma_v$ partial factor;

Figure 7: Geometrical parameters that influence the longitudinal shear capacity of the connection
The plate thickness $t_w$ (between 4 and 25mm), the distance of the headed studs to the concrete surface, the angle of the corrugation $\alpha$, and the length of the straight sections of the corrugated shape $a_l$ have lower influence, but they should be considered. The reinforcement $a_{sw}$ (stirrup per web side in cm$^2$/m) and $a_{sa}$ (additional reinforcement in cm$^2$/m) has also low influence. Finally, the strength of the steel web and the longitudinal reinforcement have a negligible effect.

3 Longitudinal shear strength of the connection with 2 layers of studs

3.1 Introduction

Based on the previous work$^1, 7$ the FE analysis of an innovative composite girder section with 2 layers of studs has been developed, Figure 8 and Figure 9. The girder is assumed to present an enhanced longitudinal shear strength thanks to the larger embedment of the corrugated steel web and the double layer of horizontal studs.

Nonetheless, this type of section has not yet been studied and there are no related experimental results in the bibliography. Within the framework of the research project P978$^{11,12}$ the assessment of its longitudinal shear resistance by means of numerical analysis has been carried out.

3.2 Numerical Analysis

For the FE modelling of the composite section the commercial program Abaqus$^{13}$ has been used. The lack of experimental data on the composite section with corrugated steel web and 2 horizontal layers of studs leads to the development of a preliminary FE analysis of already tested sections in order to calibrate the FE model. The calibration has been performed by comparison of the FE results with the experimental data from the FOSTA projects$^{1,2}$. Once the FE model of the connection with a
corrugated steel web girder has been calibrated, it is modified to know the behavior of the new composite section (corrugated web + 2 layers of studs), Figure 10 and Figure 11.

The model consists of steel web, the studs, and the steel plate for the load application, the concrete slab and the reinforcing bars. The welding of the studs to the steel web is achieved through a tie constrain, the plate for the load application is tied to the steel web in a similar way. The interaction between the studs and the concrete is introduced by means of a general contact, identifying the different surface pairs (master-slave surface) between the stud and the surrounded concrete. In the normal direction a ‘hard contact’ model is used, whereas in the tangential direction a Coulomb friction model is employed, with a friction coefficient of 0.5. The reinforcement is defined as an embedded region into the concrete (host region). Solid elements (C3D8R) are used for all the parts of the section except for the reinforcing bars that were defined as truss elements (T3D2).

The properties of the steel and the concrete are taken from the experiments. For the plastic behavior of the steel, Von Mises is used, and for the concrete performance, Damaged Plasticity Concrete (DPC).

Firstly, an increase of the height embedment and concrete thickness is simulated; secondly, an additional layer of studs is included in the model (Figure 12, Figure 13). Therefore, the influence of the embedment and of the additional layer of studs could be evaluated separately.

The three following composite sections are simulated, all of them with corrugated steel web:

- **FEM-250-8**: connection with one layer of studs, 250 mm concrete slab thickness and 8 mm steel web thickness, Figure 12.
- **FEM-350-16-1STUD**: connection with one layer of studs, 350 mm concrete slab thickness and 16 mm steel web thickness, Figure 13.
- **FEM-350-16-2STUDS**: connection with two layers of studs, 350 mm concrete slab thickness and 16 mm steel web thickness, Figure 14.
Each composite section is calculated for 4 different concrete resistances. In the Figure 15 the curves load-slippage achieved are shown. The higher stiffness and strength attributed to the thicker slab and larger embedment can be observed, as well as the further strength increase due to the second layer of studs plus the larger embedment.

![Figure 12: FE Model FEM-250-8](image)

![Figure 13: FE Model FEM-350-16-1STUD](image)

![Figure 14: FE Model FEM-350-16-2STUDS](image)

![Figure 15: Load-slippage curve from the FE Models for different concrete strengths](image)

**3.3 Longitudinal shear strength based on existing design equations**

In this section the resistance of the connection under longitudinal shear load for the three different composite sections described is evaluated by using the equation formulated by Raichle\(^7,8\), (Eq.1).
(Eq.1) only considers one layer of horizontal studs and up to now there is not any expression to calculate the capacity of a connection with more than one layer of studs. In order to calculate the resistance attributed to the second layer of studs (the stud embedded in the lower concrete slab, the prefabricated concrete slab) an approach is done by using the equations for the shear resistance of headed studs in DIN 18800-514 and EN 1994-1-115 (Eq.2 and 3). The values derived from using the steel capacity of the stud and the concrete capacity of the connection are assessed. For the concrete capacity of the connection, the German code DIN 18800-514 (Eq.3 a) uses a more conservative factor 0.25 than EN 1994-1-115, 0.29, (Eq.3 b). Furthermore, the failure due to the splitting forces for studs close to the edge included in EN 1994-2 Annex C16 (Eq.4) is also considered.

\[
P_{Rd} = \frac{0.8 \cdot f_u \cdot \pi \cdot d^2 / 4 \cdot n_s}{\gamma_v \cdot L}
\]  
\[
f_u \text{ ultimate strength of stud, not higher than 450 N/mm}^2; \\
d \text{ diameter of the headed } 16 \text{ mm} \leq d \leq 25 \text{ mm}; \\
n_s \text{ number of headed studs; L length of the connection;}
\]

\[
P_{Rd} = \frac{0.25(0.29) \cdot \alpha \cdot d^2 \sqrt{f_{ck} \cdot E_{cm}} \cdot n_s}{\gamma_v \cdot L}
\]

\[
\alpha = 0.2 \left( \frac{h_{sc}}{d} + 1 \right) \text{ for } 3 \leq h_{sc}/d \leq 4; \alpha = 1 \text{ for } h_{sc}/d > 4
\]

\[
f_{ck} \text{ characteristic cylinder strength of the concrete N/mm}^2; \\
h_{sc} \text{ overall height of the headed stud } \geq 100 \text{ mm}
\]

\[
P_{Rd,L} = \frac{1,4 \cdot k_v \cdot (f_{ck} \cdot d \cdot a_r^{'})^{0.4} \cdot (a/s)^{0.3}}{\gamma_v}
\]

\[
a_r^{'} \text{ effective edge distance } a_r^{'} = a_r - c_v - \phi_s / 2 \geq 50 \text{ mm; } \\
k_v \text{ 1.0 for shear connection in an edge position, 1.14 for a middle position; } \\
d \text{ diameter of the headed stud } 19 \text{ mm} \leq d \leq 25 \text{ mm; } \\
h \text{ overall height of the headed stud } h/d \geq 4; \\
a \text{ horizontal spacing of studs with } 110 \leq a \leq 440 \text{ mm; } \\
s \text{ spacing of the stirrups with } a/2 \leq s \leq a \text{ und } s/a_r^{'} \leq 3; \\
\phi_s \text{ diameter of the stirrups with } \phi_s \geq 8 \text{ mm; } \\
\phi_l \text{ diameter of the longitudinal reinforcement with } \phi_l \geq 10 \text{ mm; } \\
c_v \text{ vertical concrete cover in mm;}
\]

The total shear capacity of the connection for the composite section with the corrugated sheet and the two layers of studs is consequently determined by adding to the shear capacity calculated by (Eq.1), the strength from (Eq.2, Eq.3 and Eq.4). The results in comparison with the FE values are analysed.
for each alternative, Figure 16. It is concluded that for a concrete resistance lower than 48 MPa the capacity of the second layer of studs determined by the concrete failure is lower and more critical than the steel. However, for concrete resistance 48 MPa and 58 MPa, the steel resistance is lower and limits the connection capacity. The longitudinal shear strength calculated by considering splitting failure in the concrete gives values higher than the obtained with the FE model. The reason is given by three factors: the distance from the stud to the edge or lower concrete surface, the reinforcement and the dimensions of the stud, that do not lead to splitting forces. For the comparison with the FE model, characteristic values are used instead of design ones, so, the partial factor is $\gamma_v=1$.

In Figure 16 it can be observed that for lower concrete resistance, the equations give a lower estimation than the FE model and therefore, conservative value of shear capacity of the connection, whereas for 48 and 58 MPa, the use of equations are non-conservative.

In Figure 17 the values for the longitudinal shear strength of the three described composite sections calculated through the equations, are shown. For the second layer of studs the equation that gives closest value to the FE estimation is used. It can be concluded that in the FE model the increase on the connection capacity due to the increase of the concrete resistance is not so important as when it is calculated through the equations. The largest differences between the longitudinal shear resistance given by the FE model and the equations, correspond to the concrete with higher concrete resistance 58 MPa, which requires a specific analysis.

The Table 1 summarizes the values and the differences for the longitudinal shear strength of the connection between equation and FE calculation for the three composite sections under analysis. The differences on average were of 12% for the composite section of 250 mm slab thickness, 7.5% for
the concrete slab thickness of 350 mm and 4.3% for the composite section with two layers of studs and the concrete slab thickness of 350 mm. The biggest deviation in percentage (of resistance given by the FE model) appear for concrete of compression resistance 58 MPa, as it has been already pointed out. For concrete resistance of 38 MPa and 48 MPa the values of the shear capacity on the connection from the FE model and the equations are quite similar.

Table 1: Comparison of the shear capacity from FE Model and design equations

<table>
<thead>
<tr>
<th></th>
<th>Longitudinal shear strength $P_t$ (KN/m)</th>
<th>Equation</th>
<th>250_8</th>
<th>350_16_1STUD</th>
<th>350_16_2STUDS</th>
</tr>
</thead>
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<td>$f_c$ 38MPa</td>
<td>1961</td>
<td>2639</td>
<td>3842</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$f_c$ 48MPa</td>
<td>2257</td>
<td>3036</td>
<td>3853</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$f_c$ 58MPa</td>
<td>2528</td>
<td>3401</td>
<td>4217</td>
</tr>
<tr>
<td>FE Model</td>
<td></td>
<td>$f_c$ 28MPa</td>
<td>1731</td>
<td>2538</td>
<td>3231</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$f_c$ 38MPa</td>
<td>1885</td>
<td>2692</td>
<td>3654</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$f_c$ 48MPa</td>
<td>2000</td>
<td>2885</td>
<td>3808</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$f_c$ 58MPa</td>
<td>2019</td>
<td>3115</td>
<td>3846</td>
</tr>
<tr>
<td>Comparison FE Model - Equation</td>
<td>$((P_t, Model - P_t, Eq) / P_t, Model)$</td>
<td>Average</td>
<td>11.94%</td>
<td>7.46%</td>
<td>4.23%</td>
</tr>
</tbody>
</table>

Finally, the enhancement achieved by the embedment and by the embedment plus the additional layer of studs was assessed. From the results of the FE models, the average increase of the longitudinal shear capacity thanks to the embedment was 47%, and with the second layer of studs 90%, Table 2.

Table 2: Strength enhancement due to the embedment and the additional layer of studs (FE Model)

<table>
<thead>
<tr>
<th></th>
<th>FE Model- Longitudinal shear strength $P_t$ (KN/m)</th>
<th>Embedment (Concrete slab thickness)</th>
<th>Embedment + 2STUDS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>250_8 350_16_1STUD 350_16_2STUDS</td>
<td>250_8/350_16 1STUD</td>
<td>250_8/350_16 2STUDS</td>
</tr>
<tr>
<td>$f_c$ 28 MPa</td>
<td>1731 2538 3231</td>
<td>47%</td>
<td>87%</td>
</tr>
<tr>
<td>$f_c$ 38 MPa</td>
<td>1885 2692 3654</td>
<td>43%</td>
<td>94%</td>
</tr>
<tr>
<td>$f_c$ 48 MPa</td>
<td>2000 2885 3808</td>
<td>44%</td>
<td>90%</td>
</tr>
<tr>
<td>$f_c$ 58 MPa</td>
<td>2019 3115 3846</td>
<td>54%</td>
<td>90%</td>
</tr>
<tr>
<td>Average</td>
<td></td>
<td>47%</td>
<td>90%</td>
</tr>
</tbody>
</table>

4 Conclusions

This paper analyses the longitudinal shear strength of the connection in a composite bridge girder with corrugated web and horizontally lying studs. The longitudinal shear capacity of the connection with one layer of studs was studied by Raichle\textsuperscript{7,8}, experimentally and numerically, giving the formulation of an equation as a result, where the concrete resistance, the length of the steel embedment and the friction coefficient were the most important parameters.
In the case of the section with double layers of studs and a prefabricated concrete slab, the shuttering is no longer necessary and that means a reduction in times of construction and a better final appearance. In addition, there is a material saving due to the enhanced composite action of the steel and the concrete. The innovative design was analyzed by means of a sophisticated numerical model based on previous works at the University of Stuttgart. Considering the results of the FE model, the increase of the strength attributed to the larger embedment of the corrugated sheet is around 47%. The combined effect of a larger steel web embedment together with an additional layer of studs produces an enhancement of about 90% the maximum load of the basic case with one layer of studs.

The application of the equation formulated by Raichle\textsuperscript{7,8} was also assessed to calculate the capacity of the connection with the two layers of studs in combination with the equations from standards for the additional headed stud. The sum of these two resistances is compared with the resistance obtained in the FE models. It is concluded that, for lower concrete strength (<48 MPa), a good approach of the longitudinal shear capacity of the connection is obtained by adding the resistance of a stud with concrete failure by EN 1994-1-1. Nonetheless, for concrete strength beyond 38 MPa, the estimation of the resistance of the connections is better adding the capacity related to the steel failure of the stud.

5 Acknowledgements

The authors would like to express their sincere gratitude to the research association FOSTA for the funding of the research that is financed over the AiF within the development program for industrial community research and development (IGF) from the Federal Ministry of Economic Affairs and Energy (BMWi) based on a decision of the German Bundestag for the help provided through the projects P645, P843 and P978.

References:


13. ABAQUS/CAE 6.13-5 Dassault Systèmes Simulia Corp., Providence, RI, USA.


EXPERIMENTAL ASSESSMENT OF AN INNOVATIVE CORBEL

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ABSTRACT

Peikko PCs corbel is an innovative product used to create vertical supports between reinforced concrete columns and concrete or steel-concrete composite beams. When used to support reinforced concrete beams, the reaction of the beam is transferred to the corbel through a steel insert (PC beam shoe) integrated in the beam. When used with steel-concrete composite beams, the reaction from the beam is transferred by the vertical pressure of the beam to the PCs corbel. PCs corbel consists of a corbel part supporting the beam, and a column part anchoring the loads from the beam to the column. The column part consists of a system of vertical and horizontal short headed anchors. After removal of the column’s formwork the corbel part and the column part are bolted together.

A series of experimental tests has been performed to assess the performance of the anchorage part of the PCs corbel. The PCs corbels have been cast into concrete columns reinforced against bending and shear failure. Supplementary reinforcement has been used to improve the anchorage capacity of the headed anchors of the PCs corbel. PCs corbels were continuously loaded by combined shear and tensile loading which simulate the reaction of the beam on the corbel (transverse tensile load corresponding to 20% of the shear load in accordance with DAfStb Heft 5321). The external loads were applied by two hydraulic jacks installed in two perpendicular frames. The tests demonstrated the potential failure modes and failure loads of the PCs corbels cast into slender concrete columns. The tests have been used among others for German DIBt approval Z-21.8-20762.

1 Introduction

A typical arrangement of a beam to column joint using Peikko PCs corbels is shown in Figure 1. The beams are equipped with an end plate (steel beams) or a steel insert (concrete beams) with a provision that fits to the shape of PCs corbel. The transfer of vertical load from the beam to the column is provided by the pressure of the end plate or steel insert on the PCs corbel.

The corbel plate is attached to the column part of the PCs corbel through an indented contact surface consisting of horizontal ribs. Transverse bolts are used to prevent the separation of the contact surface by tightening the corbel plate to the column part.
Standardized models of the PCs corbels are pre-designed to carry vertical loads from 300kN to 1000kN. While having to anchor relatively big loads into the column, the column part of the PCs corbel also needs to remain compact to avoid the congestion of reinforcement in the column. For that reason, the anchorage of the PCs corbel is consisting of a combination of short horizontal and vertical headed anchors. PCs corbel has been developed by Peikko Group in the beginning of 2000’s and so far, used principally in countries of northern Europe. In several countries, the properties of the system have been assessed by national approvals\textsuperscript{3,4,5,6}. Within these approvals, the applicability of the PCs corbel has been conditioned by using a relatively big amount of supplementary reinforcement, designed in accordance with the European technical specification CEN/TS 1992-4-2\textsuperscript{7} (Figure 2a). In Germany, PCs corbel has been in the past approved on project basis under the condition that short headed bars are replaced by hooks designed in accordance with EN 1992-1-1\textsuperscript{8} (Figure 2b). This modification has been adopted to overcome the uncertainties concerning the resistance of short headed bars.
The solutions shown in Figure 2a and 2b are technically feasible, on the other hand they penalize the competitiveness of the PCs corbel. The large amount of supplementary reinforcement shown in Figure 2a makes the installation of the PCs corbel in the precast factory quite laborious. The alternative in Figure 2b makes difficult the standardization of the product and has a negative impact on its manufacturing time and cost.

The design methods for short headed bars with supplementary reinforcement under tensile loads have been researched by several authors over the past couple of years\textsuperscript{9,10,11,12}. The cited research works conclude that the design methods of the European specification CEN/TS 1992-4-2\textsuperscript{7} are very conservative, especially for the case of headed bars with a relatively short embedment depth. Tensile tests performed on headed bars comparable to those used in PCs corbels\textsuperscript{13} confirmed that such short bars coupled with supplementary reinforcement may anchor loads significantly higher in comparison to loads predicted in accordance with CEN/TS 1992-4-2\textsuperscript{7}.

A research program with the goal to overcome the uncertainties about the performance of the anchorage part of PCs corbel has been conducted during years 2016 and 2017. The research program consisted of full scale tests of PCs corbels cast into concrete columns and loaded by shear and tensile loads. The amount of supplementary reinforcement used to tie the PCs corbel to the column in the test specimen (Figure 2c) was significantly lower in comparison to the amount of supplementary reinforcement specified in the available technical approvals\textsuperscript{3,4,5,6} (Figure 2a).

## 2 Anchorage of PCs corbel in concrete column

Headed bars are nowadays being increasingly used as anchorage items in concrete. Their main benefit consists in the fact that they lead to more compact solutions in comparison to traditional anchorage techniques such as bends or hooks. The Eurocode EN 1992-1-1\textsuperscript{8} does not include explicit design methods for headed bars used to anchor tensile forces in columns. For that reason, the performance headed bars in Europe is most typically assessed in accordance with CEN/TS 1992-4-2\textsuperscript{7}. With short headed bars such as those used in PCs corbel, CEN/TS 1992-4-2\textsuperscript{7} typically limits the capacity of the anchorage by the resistance of the concrete surrounding the headed bar against the concrete cone failure. This resistance might be increased by using supplementary reinforcement designed and detailed to tie the potential breakaway cone to the rest of the concrete body. The supplementary reinforcement is typically provided under the form of stirrups. The technical specification CEN/TS 1992-4-2\textsuperscript{7} limits the capacity of the anchor reinforcement with the following design value of resistance:

\[
N_{Rd,a} = \sum_{\pi} l_1 \cdot \pi \cdot d_s \cdot f_{bd} \cdot \frac{\alpha}{1}
\]

where

- \(l_1\) is the lap length between the stirrup and the headed bar
- \(d_s\) is the diameter of the stirrup
- \(f_{bd}\) is the bond strength of concrete
$\alpha = 0.7$ is an empirical factor taking account of the bended shape of stirrup in the concrete cone.

Within the PCs corbel, the horizontal headed bars are designed to anchor tensile forces resulting from the eccentric position of load into the column. The vertical headed bars are assumed to anchor the vertical load in the column. Within existing approvals of PCs corbel, stirrups n1 (Figure 3) are designed in accordance with methods of CEN/TS 1992-4-2 to prevent the concrete cone failure around the horizontal headed bars. Stirrups n2 (Figure 3) are designed as tie reinforcement to resist local pressure induced by the vertical headed bars in accordance with EN 1992-1-1.

Figure 3: Load distribution model used for the design of PC corbel in existing approvals

3 Full scale tests of PCs corbels

The schematic representation of tests performed on PCs 3 corbels is shown on Figure 4. The PCs 3 corbel has been cast in a concrete column with section 280x280mm. The targeted class of concrete has been C35/45. The concrete specimen has been reinforced with normal and shear reinforcement designed to prevent the failure of the concrete specimen outside of the anchorage zone of the PCs corbel. The supplementary reinforcement consisted of two $\phi 10$mm stirrups placed around the short headed bars and one $\phi 10$mm stirrup placed in the opening corner of the vertical headed bar. The specimen has been attached to the floor of the testing laboratory and simultaneously loaded by hydraulic jacks in two perpendicular directions. The force applied in horizontal direction simulates the shear load on the corbel; in vertical direction, the corbel has been loaded by a tensile force corresponding to 20% of the applied shear force.
The specimen has been loaded up to failure that was characterized by the opening of inclined cracks in the anchorage area of the PCs corbel (Figure 5).

The load displacement curve shown on Figure 6 illustrates the ductile behavior of the PCs corbel prior to failure. Figure 6 also illustrates that the failure of the test specimen is associated with the yielding of the horizontal headed bar. The stress in the headed bar is estimated as:

$$\sigma = E_a \cdot \varepsilon \leq f_{ym}$$  \hspace{1cm} (2)

where

- $E_a = 200 \; 000 \; MPa$ is the modulus of elasticity of reinforcement steel
- $\varepsilon$ is the strain in the horizontal headed bar measures with strain gauges
\( f_{ym} \) is the mean yield strength of the reinforcement steel from which the headed bar is fabricated.

**Figure 6: Load-displacement and stress-displacement behaviour of the corbel in test PCs 3-1**

### 4 Assessment of test results

Three identical tests analogous to the test PCs 3-1 presented in this paper have been produced for each standard model of PCs corbel. The ultimate loads recorded in the tests have been used to determine the characteristic value of resistance of PCs Corbels by a statistical evaluation using a 5 \%-fractile in accordance with Annex D of EN 1990\(^{14}\) with a confidence level of 75 \%. The design values of resistance have been determined using a partial safety factor \( \gamma_c = 1,5 \) for concrete failure. This method of assessment provides a global safety factor comparable to the design models of beam to column joints formulated in reference Roeser\(^{15}\) and adopted in DIN 1045-1\(^1\).

The design values of resistances have been approved by the technical approval Z-21.8-2076\(^2\) issued by DIBt. The resistances of PCs 3 corbels validated by the DIBt approval are compared with resistances approved in earlier approvals of PCs corbels in Table 1. The resistances approved by the approval Z-21.8-2076\(^2\) are slightly smaller, on the other hand the approval requires a significantly smaller amount of supplementary reinforcement in comparison to previously available approvals.

**Table 1: Comparison of resistances of model PCs 3 in existing approvals and new test based approval Z-21.8-2076\(^2\)**

<table>
<thead>
<tr>
<th></th>
<th>Approvals(^3,4,5,6)</th>
<th>Z-21.8-2076(^2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design value of resistance</td>
<td>355 kN</td>
<td>333 kN</td>
</tr>
<tr>
<td>Number of stirrups n1</td>
<td>6φ10mm</td>
<td>2φ10mm</td>
</tr>
<tr>
<td>Number of stirrups n2</td>
<td>3φ10mm</td>
<td>1φ10mm</td>
</tr>
</tbody>
</table>
5 Conclusion

The research project summarized in the present paper has been used to provide arguments for the assessment and approval of an innovative anchorage system used in the Peikko PCs corbel. The tests demonstrated that the PCs corbels fulfills its structural function even when used with a relatively small amount of supplementary reinforcement. The design by testing proved to be a reliable and efficient method for the assessment of innovative building products, especially in the absence of reliable and cost-efficient design methods for the assessment of their performance.

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5. Plum, C.M. “Statement, use of PCs Corbel in Denmark”. 2016, Allerood, Denmark.


INFLUENCE OF STRUCTURAL STEEL ARRANGEMENT ON RESISTANCE OF COMPOSITE MEMBERS

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²ISISE, Department of Civil Engineering, University of Minho, Portugal
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ABSTRACT

This research is designed to study the behavior of composite members. A new arrangement configuration of structural steel on composite members is described. The concept of this configuration is based on using structural steel instead of conventional longitudinal steel reinforcement to provide higher confinement of concrete. The experimental program was executed by the following four different configurations of reinforcing steel: 1) reference member with conventional longitudinal steel reinforcement, 2) composite member with I-shape structural steel reinforcement, 3) composite member with T-shape structural steel reinforcement (configuration 1) and 4) composite member with T-shape structural steel reinforcement (configuration 2). However, the amount of the reinforcing steel ratio was equalized. The experimental results were presented and discussed in order to investigate the behavior of the composite members. Moreover, the effectiveness of the steel arrangement configurations was demonstrated by Finite Element Method (FEM) capable of simulating the confinement model. The efficiency of each configuration was assessed in order to propose an alternative solution for using the composite members combined with structural steel in the building.

1 Introduction

Composite members are extensively used in many modern buildings in recent decades. There are several advantages for using the composite members such as ease of installation and casting of concrete¹. The composite members can be fabricated in a manufacture and readily delivered to a site work. This advantage can reduce the working time of labors and the total cost of construction projects.

Many researches have been carried out on behavior of composite structures. For example, Chen and Lin² studied the behavior of concrete encased steel composite stub columns. Structural steels were arranged in different forms to determine the effective confinement of concrete core. It was clearly seen that the configurations of the structural steel arrangement inside section can provide highly confined concrete in the core. Lee³ presented the advantages of composite members compared to conventional reinforcement members. It was found that the use of composite members has a better performance than the conventional reinforcement members in terms of cost efficiency, construction
duration, quality of work and safety. However, the configuration of structural steel arrangement in composite section is still limited. It needs to be further investigated to provide an extensive experimental database of composite members.

This research, a new configuration of structural steel arrangement on composite members is studied. The concept of this configuration is based on using structural steel instead of conventional longitudinal steel reinforcement to provide higher confinement of concrete. The experimental program is presented and discussed. Moreover, the experimental program is simulated by using a finite element based program (ABAQUS). Finally, the results obtained from the simulations were verified and compared with the experimental results.

2 Experimental Program

2.1 Specimen Details
All specimens in this research have a cross section of 400x400 mm$^2$ and length of 3000 mm. The experimental program was executed by the following four different configurations of reinforcing steel: 1) reference member with conventional longitudinal steel reinforcement (RC), 2) composite member with I-shape structural steel reinforcement (SRC1), 3) composite member with T-shape structural steel reinforcement (SRC2) and 4) composite member with T-shape structural steel reinforcement (SRC3). The amount of reinforcing steel ratio was equalized for all sections. The details of the sections are presented in Figures 1 and 2.

Figure 1: Types of composite members (dimension in mm)
2.2 Materials properties

2.2.1 Concrete
A ready mix concrete with the nominal strength design of 24 MPa was used. The mix proportion was prepared from the local Portland cement, local coarse and fine aggregate. The maximum size of aggregate was 25mm. The concrete was cured at the room temperature until the testing day. However, the compressive strength of concrete (from the compressive cylinder tests) was approximately 18 MPa for the reference member and 20 MPa for the composite members at the testing day.

2.2.2 Steel Reinforcement
Deformed bars and structural steels were used in this study. The deformed bars have different sizes, depends on the configurations of the steel arrangement. The diameter of the deformed bars with 12, 20 and 25 mm were used as longitudinal steel bars. The nominal yield strength of the steel bars was 530, 430 and 540 MPa for the diameter of 12, 20 and 25, respectively. For the structural steels, an I-shape and T-shape were selected for the arrangement of the sections. The structural steel I-shape of 200x100mm and weight of 23.1 kg/m was used. The T-shape structural steels were made of an original H-shape (which is cut in a haft section of the H-shape). The structural steel H-shape dimension of 150x75mm and weight of 14 kg/m were selected before cutting the half section. The nominal yield strength of the structural steels, provided by the company was 340 MPa.

2.3 Test setup
All specimens were subjected to a 3-Point bending moment loading (Figure 3) with a universal loading machine of 500KN capacity. For monitoring the tests, the strain gauges were installed at the bottom steel reinforcement at the mid-span of the specimens in order to record the strain in each loading increment. Linear transducer displacements (LVDT) were also installed to record the deformation at the mid-span. A represent of the test setup is shown in Figure 3.
3 Experimental results

3.1 Overall behavior

From the experimental tests, the maximum load carrying capacities of all members are presented in Table 1. The reference member (RC) has a higher load than the composite member with T-shape configuration 1 (SRC2), I-shape (SRC1) and T-shape configuration 2 (SRC3), respectively. However, the reference member has the higher loading capacity due to the higher percentage and tensile strength of tensile steel reinforcement at the bottom level compared to the composite members. The force versus mid span deflection of the members is shown in Figure 5.

Table 1: The summary of the maximum load and mid-deflection

<table>
<thead>
<tr>
<th>Types of members</th>
<th>Maximum load (KN)</th>
<th>Maximum Mid-Deflection (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reference member (RC)</td>
<td>440</td>
<td>24.97</td>
</tr>
<tr>
<td>Composite I-Shape (SRC1)</td>
<td>330</td>
<td>28.94</td>
</tr>
<tr>
<td>Composite T-Shape configuration 1 (SRC2)</td>
<td>345</td>
<td>23.00</td>
</tr>
<tr>
<td>Composite T-Shape configuration 2 (SRC3)</td>
<td>330</td>
<td>25.60</td>
</tr>
</tbody>
</table>

3.2 Failure modes

The failure of all specimens was dominated by concrete crushing after yielding of steel reinforcement. The propagation of inclined flexural shear cracks was mostly appeared in the reference member. The composite members have less inclination angle of the shear cracks since the structural steels inside the composite members can significantly provide a higher shear resistance capacity than the reference member. All of the failures of the members can be observed in figure 5.
3.3 Force versus strain response

The tensile strains of the longitudinal steel reinforcement are shown in Figure 6. The strains were measured at the same level of the sections. It can be seen that the composite section SRC1 had reached the yielding of the steel earlier than the other composite sections due to the less percentage of steel reinforcement ratio at the same bottom level.
Moreover, it was found that the sliding between the structural steel and the concrete was occurred. For example, the strains of the bottom level at mid span of the composite section SRC3 were not the same. In theory, the strain distribution at the same level should be equal for the assumption of the perfect bond between the concrete and the reinforced steel, but the obtained results from the experimental test were different. Considering at a constant load level, the strain of the steel bar has a higher strain value than the structural steel. It can be concluded that the sliding of the structural steel was generated during the applied loading.

**Figure 6**: Force versus strain of the tensile longitudinal steel at mid span

**Figure 7**: Comparison the strains at the same level

### 4 Finite Element Analysis Modelling

The simulation of the experimental tests was provided by ABAQUS Finite Element Program which is capable to simulate the damage for reinforce concrete element. The material models are presented in the following
4.1 Concrete material model

The cementitious matrix was simulated with the “concrete damage plasticity” material model (CDP) that is readily available in ABAQUS. This model is based on the theory proposed by Drucker-Prager, then later developed by Lubliner, Lee and Fenves (ABQUS’s manual)\(^4\). The model includes the effects from compressive behavior, tensile behavior, and damage conditions of the material. More details can be found in ABQUS’s manual\(^4\). The input data for the concrete material model are presented in the following.

4.1.1 Uniaxial compressive behavior of concrete

The uniaxial compressive stress-strain behavior of concrete was obtained by the work of Mander et al.\(^5\). The stress strain model is presented in Figure 8. The compressive stress is given by

\[
f'_c = \frac{f'_{cc} x r}{r - 1 + x'}
\]  

(1)

Where \( f'_{cc} \) is the compressive strength of confined concrete, in this study \( f'_{cc} \) is defined as \( f'_{co} \) which is represented the unconfined concrete.

\[
x = \frac{\varepsilon_c}{\varepsilon_{cc}}
\]  

(2)

where \( \varepsilon_{cc} \) is the compressive concrete strain, determined by

\[
\varepsilon_{cc} = \varepsilon_{co} \left[ 1 + 5 \left( \frac{f'_{cc}}{f'_{co}} - 1 \right) \right]
\]  

(3)

\[
r = \frac{E_c}{E_c - E_{sec}}
\]  

(4)

\[
E_c = 5000\sqrt{f'_{co}}
\]  

(5)

\[
E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}}
\]  

(6)

Figure 8: Uniaxial stress-strain of concrete
4.1.2 Uniaxial tensile behavior of concrete

The uniaxial tensile stress-strain behavior of normal concrete is represented by the work of Wahalathantri et al. The model was developed to avoid a run time convergence error after the post peak of the tensile strength of concrete. The represented model is illustrated in Figure 9.

![Figure 9: Uniaxial tensile stress strain of concrete](image)

4.1.3 Other material properties for CDP model

The other material properties of the concrete plasticity model were defined for the FEA program to calculate the concrete damage plasticity flow potential, yield surface and viscosity parameters. All parameters are described below

1. Dilation angle, $\psi$

   Dilation angle is measured in the shape of the yield surface in the meridional plane (p-q). In the other word, it is the friction angle of the material in the meridional stress plane. In this study, the dilation angle ($\psi$) of 31 degree is used.

2. Flow potential eccentricity, $\varepsilon$

   Flow potential eccentricity is a parameter to define the rate at which the hyperbolic flow potential approaches its asymptote. The default value of $\varepsilon=0.1$ is used.

3. $f_{ho}/f_{co}$

   This parameter is the ratio of initial equivalent biaxial compressive yield stress to initial uniaxial compressive yield stress. The default value of $f_{ho}/f_{co}=1.16$ is used.

4. $K_e$

   This parameter is the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian. The default value of $K_e=2/3$ is used.

5. Viscosity parameter, $\mu$

   The viscosity parameter is used for the visco-plastic regularization of the concrete constitutive equations in Abaqus/Standard analyses. The default value of $\mu=0$ is used.
6. Damage parameters, $d_c$ and $d_t$

The damage parameters are obtained from the work of Wahalathantri et al.\(^6\). The compressive damage parameter $d_c$ is calculated from the ratio of the inelastic strain to the total strain. In the same way, the tensile damage parameter $d_t$ is obtained from the ratio of the cracking strain to the total strain.

<table>
<thead>
<tr>
<th>Concrete Damage Plasticity Model</th>
<th>Concrete Damage Plasticity Model</th>
<th>Concrete Damage Plasticity Model</th>
<th>Concrete Damage Plasticity Model</th>
<th>Concrete Damage Plasticity Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dilution angle, $\psi$</td>
<td>Eccentricity, $\varepsilon$</td>
<td>$f_{so} / f_{co}$</td>
<td>$K_c$</td>
<td>Viscosity parameter, $\mu$</td>
</tr>
<tr>
<td>31</td>
<td>0.1</td>
<td>1.16</td>
<td>2/3</td>
<td>0</td>
</tr>
</tbody>
</table>

### 4.2 Steel reinforcement constitutive model

The stress-strain relationship for steel rebar used in the model was obtained from an actual tensile test of the steel rebar by the universal testing machine (UTM). The average value of the stress-strain data sets from three test trials were used as the input of the plasticity function in ABAQUS. Moreover, the tensile stress-strain model of the structural steel was obtained from the AISC standard\(^7\) which has the yield strength equal 340 MPa.

![Figure 10: The input data of the tensile stress-strain of the steel bars in ABAQUS](image-url)
4.3 Description of finite element meshing and boundary conditions
Each of the components in the model was divided into equally sized elements. The element sizes were reduced after each analysis trial, until the above finer mesh provided a solution converged. The boundary conditions at the planes of symmetry were created. Moreover, all members were loaded by displacement control at the symmetric planes. Truss elements were generated for the steel reinforcing bars and 3D-solid elements were generated for the concrete and structural steels. The steel reinforcements were embedded corresponding to the degrees of freedom in concrete. It could be assumed that the steel reinforcements and the concrete were bonded perfectly.

1) Reference member (RC)
2) Composite I-Shape (SRC1)
3) Composite I-Shape configuration 1 (SRC2)
4) Composite I-Shape configuration 2 (SRC3)

![Finite element mesh of the members](image)

Figure 11: Finite element mesh of the members

5 Finite Element Analysis Results and Discussions
The results of FEM analysis are presented in Figure 12. A comparison between the finite element and the experimental results was carried out to verify the simulation models. It can be seen that the FEM results has a good acceptance for the reference member. The FEM results of the composite members were slightly different from the experimental results. Since the FEM models were assumed as a perfect bond between concrete and structural steels. But the results from the experimental tests showed that there were the slip between the surface of concrete and structural steels. According to these results for the composite members, the FEM models should be improved for an accurate prediction. An interface model should be implemented to capture the bond-slip behavior of the interface between concrete and structural steel.
In this work, the behavior of composite members had been studied. A new arrangement configuration of structural steel on composite members were carried out based on the concept of using structural steel instead of conventional longitudinal steel reinforcement. The experimental program was executed by the following four different configurations of reinforcing steel: 1) reference member (RC), 2) composite member with I-shape structural steel reinforcement (SRC1), 3) composite member with T-shape structural steel reinforcement configuration 1 (SRC2) and 4) composite member with T-shape structural steel reinforcement configuration 2 (SRC3). The results can be summarized as below.

- The conventional arrangement of longitudinal steel reinforcement provides higher load carrying capacity than the composite members due to the higher tensile strength of the steel bars and percentage of the reinforcement ratio at the same level.

**6 Conclusions**

Figure 12: Comparison of the experimental test and FEM analysis for all specimens
The influence of concrete confinement provided by the structural steel arrangements is less effective for the type of this test.

The composite members combined with structural steels can be used for an alternative solution in the case of ease of installation and casting of concrete.

FEM results have a good agreement for the prediction of the overall behavior with the reference member, compared to the experimental results. However, the simulations of the composite members need to be improved by using a constitutive model of an interface element to capture the bond-slip behavior between concrete and structural steel.

In the future, the compressive behavior of these configurations will be investigated to study the performance of the confinement of concrete. The FEM models will be improved by using an interface element between the surface of concrete and structural steel to simulate a realistic behavior of composite members.

7 Acknowledgement

The authors would like to acknowledge the support received from Chulalongkorn University with scholarships of AUN/SEED-Net program and “Stimulus Package 2 (SP2) of Ministry of Education under the theme of Green Engineering for Green Society” of Thailand.

References:


EFFECTS OF IMPOSED DEFORMATIONS AT THE INTERFACE OF
END REGIONS OF STEEL-CONCRETE COMPOSITE BEAMS

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ABSTRACT

The performance of steel-concrete composite structures may be affected by imposed deformations like long-term effects of concrete or thermal variations. The real influence of imposed deformations is a function of the interaction degree between the concrete and steel components of the structure, i.e. the stiffness of the connection. The present study deals with the effect of imposed deformations on the connection of end regions of composite beams consisting of a steel beam and a concrete slab on top, with emphasis on undesired uplift effects. An analytical model is exploited to study the local distribution of normal and tangential stresses at the connection between concrete and steel of composite beams, as well as relative tangential and normal displacements. The study makes use of the bond factor and the vertical-to-tangential stiffness ratio to characterize the connection between the steel beam and the concrete slab. In this way, different types of connection from flexible studs to stiff welded angle connectors can be analyzed just by introducing the appropriate values of the stiffness of the interface. In particular, in the present contribution an analytical approach is presented to calculate the tangential and vertical stiffness of headed shear studs. Then, analytical equations allow quantifying uplift effects at end regions. The model has been used to derive parametric diagrams which show the tensile force at the most loaded connector produced by an imposed deformation due to concrete shrinkage. The results can be used to control local stresses at the interface, which can be combined with good practice rules to prevent uplifting. In particular, it is obtained that uplift forces higher than 0.1 times the shear strength of the connector might occur, which would lead to a reduction of the resistance of shear studs by shear-tension interaction.

1 Introduction and background

Composite steel-concrete composite structures consisting of a steel beam and a concrete top slab may suffer relative interface displacements when the connection between the steel and concrete components is flexible. Under such partial interaction, the internal distribution of stresses and strains, and the whole deformability, differ from those estimated using the well-known concept of the transformed (homogenized) section¹. Knowledge to study the effect of partial shear interaction on composite beams departs from the classical approach by Newmark et al.², who considered explicitly the tangential slip between the components of the cross-section. Under the hypotheses that plane sections remain plane, equal curvatures of the two components of the section, linear behavior of materials and linear behavior of the connection, closed-form solutions can be derived for composite beams under
The introduction of the possibility of vertical separation between the components of the section is not that straightforward. Some authors have derived the differential equations to consider vertical partial interaction but numerical methods have been preferred to solve the problem, instead of the derivation of closed-form solutions.

The complexity of the problem increases in case the composite structure is subjected to indirect actions. The problem has been studied numerically by a number of authors but they have not accounted for vertical partial interaction. It has to be noted that current design codes for composite steel-concrete structures require shear connectors to be able to prevent uplift or vertical separation but, because of the difficulties involved in modeling such effects, good practice rules have been typically provided (e.g. regarding dimensions for the head of shear studs). Nevertheless, there is a number of reasons which makes clear that the quantification of uplift effects is of practical interest, e.g. 1) possibility of local problems in end regions and load introduction areas, 2) reduction of shear strength of shear connectors in presence of moderate tensile forces, or 3) excessive long-term deformations.

The quantification of uplift effects produced by indirect actions like concrete shrinkage requires accounting for the tangential and normal partial interaction of the interface of composite steel-concrete beams. Recently, Zanuy has proposed and validated an analytical model which provides closed-form equations for the (tangential and normal) relative displacements and stresses at the interface of simply supported beams. A fundamental hypothesis of the model, which differs from previous assumptions used in classical approaches like the one by Newmark et al., is that the steel and concrete components of a composite beam do not have equal curvatures. According to Zanuy, the longitudinal distribution of relative tangential slip ($s$) and vertical displacement ($y$) at the interface can be calculated as follows:

$$s(x) = \frac{-\varepsilon_{cs}L}{\lambda L} \left[ \sinh(\lambda x) - \tanh\left(\frac{\lambda L}{2}\right) \cosh(\lambda x) \right]$$

$$y(x) = \frac{M(\lambda L)}{\cosh \beta L + \cos \beta L + 2 \cos^2 \frac{\beta L}{2}} \left[ (\cos \beta L - \sin \beta L) \sinh \beta x \cos \beta x + \left(2 \cos^2 \frac{\beta L}{2} - \sin \beta L\right) \sinh \beta x \sin \beta x + \sinh (L - x) \cos \beta x \right]$$

The meaning of symbols used in the previous equations is as explained in Figure 1. The compressive concrete shrinkage strain $\varepsilon_{cs}$ is positive. It has to be noted that positive and negative values for $y$ mean penetration and separation, respectively. Corresponding tangential and normal stresses at the interface ($\tau$ and $\sigma_v$, respectively) can be calculated by:

$$\tau = Ks; \quad \sigma_v = K_v y$$

where $K$ and $K_v$ are the tangential and normal stiffness of the interface. As advised by some authors, $K_v$ should be different in tension (uplift) and compression (bearing). The typical shape of the stress
distributions at the steel-concrete interface is as represented in Figure 1, where it can be noted that uplift stresses develop near the ends.

Figure 1: Overview of notation and symbols used in the formulation. Subscripts \(c\) and \(s\) refer to concrete slab and steel section, respectively.

The analytical model (see Eqs. (1) and (2)) makes use of a so-called bond factor \(L\) which considers the partial shear interaction in a non-dimensional sense, while \(\beta L\) works similarly for partial vertical interaction. Even though the former equations may seem complex, they can be easily implemented in a data sheet.

In the previous equations, the stresses at the interface extend over the whole steel-concrete contact of width \(b\) (see Figure 1). Nevertheless, the real situation commonly consists of discrete connectors placed at a certain spacing from each other. Therefore, real shear and uplift forces at the connectors are to be calculated from \(\tau\) and \(\sigma_v\) making use of the tributary area of each connector. Accordingly, the real distribution of connectors can play a significant role in the amount of forces carried by them. Moreover, it seems necessary to establish a model to estimate \(K\) and \(K_v\) as a function of employed connecting devices. In section 3, a model is proposed to calculate the stiffness of the connection for shear headed studs. Then, the model is exploited to quantify uplift effects in practical situations.

2 Influence of partial interaction

The effect produced by concrete shrinkage on the global response of a single-span composite beam with partial shear interaction is represented in Figure 2(a), where the midspan deflection has been represented as a function of the bond factor \(\overline{L}\). The formula to calculate de midspan deflection can be found in Ref. 11. The deflection \(v\) is normalized to \(v_{hom}\), which corresponds to full interaction and it can be calculated with the transformed section method. From Figure 2(a), it can be observed that
full interaction corresponds to shear stiffness values leading to bond factors larger than 20, while smaller bond factors result in partial interaction (obviously the midspan deflection is zero when there is no connection, i.e. $\bar{\lambda}L = 0$, which would lead to free shortening of the concrete slab). The influence of the partial shear interaction on the tangential stress distribution along the steel-concrete interface is depicted in Figure 2(b), for two values of the bond factor: $\bar{\lambda}L = 8$ (partial interaction) and $\bar{\lambda}L = 25$ (full interaction). It is shown that a stiffer connection results in higher tangential stresses. The former results correspond to the composite beam plotted in Figure 2(c), consisting of a HEB-200 steel beam and a 1.00 x 0.12 m concrete slab. The free concrete shrinkage is assumed as $\varepsilon_{cs} = 450 \cdot 10^{-6}$.

![Graphs and diagrams showing midspan deflection and tangential stress distribution](image)

Figure 2: (a) Influence of partial shear interaction on the midspan deflection due to concrete shrinkage; (b) Tangential stress distribution along the interface ($\varepsilon_{cs} = 450 \cdot 10^{-6}$); (c) Geometry of the example.

An interesting observation from Eqs. (1)-(3) is that tangential stresses at the interface only depend on the shear stiffness $K$. In contrast, normal stresses depend on both the shear and normal stiffness $K$ and $K_v$. In order to study the influence of both parameters, a parametric study has been performed for the beam of Figure 2(c). The longitudinal distribution of relative vertical displacement and normal stress at the steel-concrete interface are represented in Figure 3 for two values of $\bar{\lambda}L$ (corresponding to partial and full interaction) and $K_v/K$ ratios of 0.1, 0.5, 1, 5 and 10. As it can be noted, vertical separation and uplift stress can be expected at the end region. According to Figure 3, the vertical separation results are smaller under full interaction than under partial interaction, and the contrary occurs regarding the uplift stress. In addition, the relative displacement decreases as the ratio $K_v/K$ increases, which also results in larger uplift stresses.

The critical cross-section regarding separation and uplift stress is the one at the support ($x/L = 0$); therefore, it seems convenient to focus on the results at that critical section in order to study the most unfavorable effects produced by the imposed concrete shrinkage strain. The results are represented in Figure 4 for a wider range of the bond factor and the stiffness ratio. An interesting finding is that the maximum vertical separation is obtained for a bond factor of about $\bar{\lambda}L = 2-2.5$. The maximum uplift stress increases as both stiffness $K$ and $K_v$ increase. In order to control the uplift stress and to understand whether obtained values of uplift stresses are excessive, it is convenient to have some tools to determine the values of the shear and normal stiffness of the connection. In the following section, a proposal is given to estimate stiffness provided by shear headed studs.
3 Estimation of the stiffness of the connection

3.1 Shear stiffness of shear studs

The shear stiffness provided by the connection is responsible for the bond factor $\lambda L$ and plays a big role in the development of uplift stresses at end regions of composite steel-concrete beams. A model
to estimate the tangential stiffness of shear studs is presented next. The flexibility of studs has been typically studied by means of push-out test as defined by Annex B of EC-4\(^{10}\). With such testing scheme, the whole shear force-relative slip curve of the connecting device can be obtained. Many experimental results have been reported in the scientific literature for shear studs, typically showing a nonlinear behavior like that plotted in Figure 5 for well-documented tests by Chapman & Balakrishnan\(^{12}\). Models have been proposed for the shear force-relative slip curve, such as those by Ollgaard et al.\(^{13}\) or Topkaya et al.\(^{14}\), Eqs. (4) and (5) respectively:

$$\frac{Q}{Q_u} = \left(1 - e^{-18s}\right)^{0.4}$$  \hspace{1cm} (4)

$$\frac{Q}{Q_c} = \frac{3(s/s_c)}{1 + 2(s/s_c)}$$  \hspace{1cm} (5)

where \(Q_u\) is the shear resistance of the connector, \(s\) is expressed in inches in Eq. (4), \(Q_c = 0.8Q_u\), \(s_c = d_{sc}/25\), and \(d_{sc}\) is the diameter of the stud in mm. The goodness of such models is compared with experimental results by Chapman & Balakrishnan\(^{12}\) in Figure 5. A nonlinear model for the connector is not convenient for the model referred to in sections 1 and 2, which actually focuses on the in-service condition. From their nonlinear model (Eq. (5)), Topkaya et al.\(^{14}\) have recommended the use of a secant stiffness corresponding to a slip of \(s_c\). The resulting linear behavior is plotted in Figure 5(left) in comparison with the original nonlinear model and also the initial tangent stiffness. As it can be noted, the secant stiffness at \(s_c\) provides a more flexible response than that experimentally measured for smaller slips, but Topkaya et al.\(^{14}\) and Wang\(^{4}\) have advised that the shear stiffness in a real composite beam is somewhat smaller than the one obtained in the standard push-out test, which justifies the use of the secant stiffness.

![Comparison with Topkaya et al. (2004)](image)

**Figure 5:** Shear force-tangential slip behavior of shear studs. Experimental curves from Chapman & Balakrishnan\(^{12}\) correspond to shear studs of diameter \(d_{sc} = 19\) and \(13\) mm, and height \(h_{sc} = 102\) and \(51\) mm.
Nevertheless, the determination of the secant stiffness of the connector \((K_c)\) from Eq. (5) depends on the ultimate shear resistance \(Q_u\), which is not very convenient. Therefore, a simpler model is proposed here to calculate \(K_c\). The deformed shape of a shear stud can schematized as in Figure 6(a), where it is observed that the deformation concentrates on a certain length close to the welded section. Moreover, the deformed length is affected by both bending and shear deformations. A simple formulation can be obtained by similarity of the deformed length with a beam of length \(L\) with two fixed supports subjected to a lateral displacement (Figure 6b). The corresponding stiffness can be calculated as:

\[
K_c = 12 \frac{EI}{h_e^3} \tag{6}
\]

where \(EI\) is the bending stiffness of the connector and \(h_e\) is the equivalent deformed length, which can be obtained from analysis of tests. In the present model, \(h_e\) has been estimated to provide the same secant stiffness than the model by Topkaya et al. (Eq. (5)). Thus, the estimated value of \(h_e\) is assumed to provide the real stiffness of the connector, implicitly including shear deformations, even though Eq. (6) is set for bending deformations. From the analysis of connector tests by Chapman & Balakrishnan\(^1\), it has been obtained an average equivalent length of \(h_e = 2.6 \delta_{sc}\) to produce the secant stiffness of Eq. (5).

![Figure 6: (a) Deformed shape of a shear stud; (b) Beam with fixed supports under lateral displacement.](image)

### 3.2 Normal stiffness of shear studs

The vertical stiffness of a headed stud \((K_{cv})\) comes from the resistance against separation or pull-out. Much less work has been done regarding the normal behavior of connectors than the tangential response and research has mainly focused on the ultimate tensile loading capacity rather than on the tensile stiffness. The pull-out failure mode of headed shear studs is typically given by the formation of a shear cone (Figure 7a) and existing formulations suggest that the pull-out resistance is provided by the splitting tensile strength of the lateral surface of the shear cone (see e.g. ACI 349.2R\(^1\)). Appropriate corrections to the failure surface are to be introduced in case of closely spaced studs due to overlapping of concrete shear cones (Ref.15). In order to have an estimate of the pull-out stiffness
it is here proposed to consider an equivalent concrete spring of diameter \( d_{eq} \) and length \( h_{ce} \), as depicted in Figure 7(b). The corresponding spring stiffness can be calculated as:

\[
K_v = \frac{E A_{eq}}{h_{ce}} \tag{7}
\]

where \( A_{eq} \) is the cross-section area of the equivalent spring \((\pi d_{eq}^2/4)\). The quantification of the equivalent diameter can be done by fitting experimental pull-out tests. Nevertheless, pull-out tests have mainly focused on ultimate strength and the full pull-out curve has not been typically reported in the literature. The estimation by Balakrishnan \(^{16}\) of the vertical pull-out stiffness of a headed stud of \( \frac{3}{4} \times 4 \) in \((d_{sc} = 19 \text{ mm}, h_{sc} = 102 \text{ mm}, d_h = 32 \text{ mm}, h_h = 13 \text{ mm})\) is considered as reference. From pull-out tests, Balakrishnan \(^{16}\) estimated a stiffness of \( K_{cv} = 176 \text{ kN/mm} \), which can be reproduced by Eq. (7) if the diameter of the equivalent cross-section is \( d_{eq} = 0.28(d_h + h_{ce}) \). In the following, the vertical stiffness of the connection will be calculated with that equivalent diameter. The resulting \( K_v/K \) ratio for the considered \( \frac{3}{4} \times 4 \) in stud is 1.4.

3.3 Stiffness of the connection

The tangential and normal stiffness of the connection \((K \text{ and } K_v)\) can be calculated from the shear and vertical stiffness of the connectors \((K_c \text{ and } K_{cv})\), by considering that the load carried by each single connector is spread on a tributary area of the steel-concrete interface. Considering that \( N_c \) is the number of connectors per row and \( S_c \) is the longitudinal spacing (Figure 8), the following equations can apply to calculate the connection stiffness:

\[
K = \frac{N_c K_c}{b S_c} \tag{8}
\]

\[
K_v = \frac{N_c K_{cv}}{b S_c} \tag{9}
\]
Practical examples

In order to study the influence of an imposed deformation due to concrete shrinkage on the development of uplift stresses at the end of a composite steel-concrete beam, a parametric example is presented below. The geometry of Figure 2(c) is considered in the analysis. The free concrete shrinkage is taken as $450 \times 10^{-6}$. Shear studs of $\frac{3}{4} \times 4$ in ($d_{sc} = 19$ mm, $h_{sc} = 102$ mm, $d_h = 32$ mm, $h_h = 13$ mm) and $\frac{1}{2} \times 2$ in ($d_{sc} = 12.7$ mm, $h_{sc} = 50.8$ mm $d_h = 19$ mm, $h_h = 5$ mm) are evaluated. The number of connectors per row are $N_c = 1$ and 2. In the results, the influence of the longitudinal spacing is studied ($S_c \geq 5d_{sc}$).

The results corresponding to $\frac{3}{4} \times 4$ in studs are represented in Figure 9. The graphics of Figure 9(a-b) can help in understanding the stiffness factors achieved with $\frac{3}{4} \times 4$ in studs: the stiffness ratio $K_v/K$ decreases from 2.4 to 1.6 when 2 studs per row are used instead of 1 stud, due to the overlapping of shear cones because of the small width of the top flange of the steel profile. The bond factor is clearly dependent on the longitudinal spacing of studs. Moreover, bond factors corresponding to partial interaction are obtained with 1 stud per row, and higher bond factors are obtained with 2 studs per row (full interaction is attained for a longitudinal spacing shorter than 0.24 m = $12d_{sc}$). The connection degree $\eta$ is represented in Figure 9(c). According to EC-4 (2004), $\eta \geq 1.0$ corresponds to full shear connection, i.e. the connectors are able to resist the total design shear force in ultimate limit state. According to Figure 9(c), full connection would be achieved with a longitudinal spacing smaller than 0.2 m or 0.4 m for 1 or 2 studs per row, respectively. Finally, the uplift effects at the end of the beam are represented in Figure 9(d) in terms of the highest tensile force carried by the connectors ($T_{max}$). Such tensile force is calculated from the maximum uplift stress at the end of the beam multiplied by the tributary area of a single connector. The tensile force $T_{max}$ is normalized in Figure 9(d) over the ultimate shear strength of a connector, calculated as follows:

$$Q_u = \min \left[ 0.8 f_u \pi d_{sc}^2 / 4; 0.29 \alpha d_{sc}^2 \sqrt{f_y E_c} \right], \quad \alpha = 0.2 \left( h_{sc} / d_{sc} + 1 \right) \leq 1.0$$

The presentation of the highest tensile force in normalized form allows understanding the significance of uplift effects, since it is advised by the codes that the ultimate shear capacity of a stud is reduced when the tensile force is larger than 0.1$Q_u$. According to Figure 9(d), the tensile force
would be higher than $0.1Q_u$ if 1 stud per row is used, and it would be smaller than $0.1Q_u$ only if 2 studs per row are placed at a longitudinal spacing closer than 0.17 m ($8.9d_{cu}$).

Figure 9: Effects of concrete shrinkage of $\varepsilon_{cs} = 450 \cdot 10^{-6}$. $\frac{3}{4} \times 4$ in studs: (a) Stiffness ratio; (b) Bond factor; (c) Connection degree; (d) Uplift forces.

The results of the study with $\frac{1}{2} \times 2$ in studs are represented in Figure 10. As it can be noted, the stiffness ratio, the bond factor and the connection degree decrease with respect to those obtained with $\frac{3}{4} \times 4$ in studs. The consequence in terms of uplift effects at the end of the composite beam is a higher tensile force carried by the most loaded connector. According to Figure 10(d), the maximum tensile force would be higher than $0.1Q_u$ for all considered combinations, which would produce a reduction of the shear capacity of the studs.

5 Conclusions

In this contribution, an analytical model has been presented to analyze the local relative displacements and stresses at the steel-concrete interface of composite beams under an imposed concrete shrinkage strain. This contribution includes a proposal to obtain the tangential and vertical stiffness of the connection as a function of the geometry and material details of shear studs, but the model is so general that it can be used for any connector type just by introducing its particular stiffness values. The model shows that the most unfavorable uplift effects develop at the end supports. The interest of the model is that it can be easily used in a simple data sheet to quantify
uplift effects. A parametric study has been included in this contribution and it has been shown that tensile forces higher than 0.1 times the theoretical shear strength of considered studs can develop in the most unfavorable section, which could result in a reduction of the resistance of the studs according to current codes\textsuperscript{10}.

![Graphs showing effects of concrete shrinkage and connector spacing on stiffness ratio, bond factor, connection degree, and connector tensile force.](image)

Figure 10: Effects of concrete shrinkage of $\varepsilon_{cs} = 450 \cdot 10^{-6}$. $\frac{1}{2} \times 2$ in studs: (a) Stiffness ratio; (b) Bond factor; (c) Connection degree; (d) Uplift forces.

### 6 Acknowledgement

The support of the Dept. Continuum Mechanics and Structures at UPM and Fundación Agustín de Betancourt are gratefully acknowledged.

### References:


Composite dowels are efficient, innovative shear connectors consisting of interlocking steel and concrete dowels. They can be used to transfer shear forces between the concrete slab and the steel section in composite beams. In regions with negative bending moment, where concrete cracking occurs, the shear capacity of composite dowels with pry-out failure is significantly reduced. Concrete cracks appear as discrete discontinuities in the pry-out cone, which induce a pry-out cone detachment. In the cracks, the shear stresses are transferred via aggregate interlock. However, well-known design models for composite dowels neglect the impact of transversal cracking and, hence, overestimate their shear capacity in cracked concrete. To resolve that deficit, the present paper proposes a mechanical model for assessing the pry-out capacity. The novel approach agrees well with experimental investigations. The model is validated by a comprehensive test data base.

1 Introduction

In composite steel and concrete structures, composite dowels can be used to transfer shear forces between the concrete slab and the steel section. Composite dowels have been developed as an alternative for headed stud connectors. They are produced by plasma or laser cutting. The cutting torch burns regular open (e.g. puzzle- or clothoid-shaped) or closed recesses (e.g. perfobond shape) immediately into the web of a steel beam or into a steel strip, which is subsequently welded to the upper flange of the steel beam. After encasing the strip with concrete, the vertically embedded steel dowels and the interstitial concrete dowels ensure a structural, interlocked connection. Composite dowels combine high shear capacity and sufficient deformation capacity and yield very resource efficient composite members, especially in composite sections with single-flange steel beams, where the relative ineffective steel part near the plastic neutral axis is reduced. Under static loading, a concrete pry-out failure of the composite dowel can occur, while the steel dowel may fail due to combined shear and bending stresses.

Due to their compact geometry, composite dowels are perfectly suitable for the use in slender concrete slabs. Here, the low embedment depths usually lead to a pry-out failure in the ultimate limit state. Design models for the concrete pry-out failure of composite dowels have been proposed by ZAPFE, SEIDL and HEINEMEYER. These models were recently consolidated to a general technical
approval for composite dowels, as the use of composite dowels is currently not embodied in international standards (for example EC4\textsuperscript{7}). Until now, the experimental and theoretical investigations focused on the application of composite dowels in concrete compression chords, where concrete cracking is omitted. But if the concrete slab is exposed to tensile stresses – for example near the interior supports of continuous composite beams - transverse cracking occurs. Through this, the composite dowels’ shear capacity is significantly affected\textsuperscript{8}. So far, the well-known design models do not account for the internal carrying mechanisms of composite dowels in cracked concrete. Therefore, they usually overestimate the shear capacity in cracked concrete. To resolve that safety deficit, the present article proposes a mechanical model for the assessment of the pry-out capacity of composite dowels in cracked concrete.

2 Phenomenology and test results

Former shear tests\textsuperscript{8,9} from the literature indicate that transverse concrete cracks significantly reduce the shear capacity of composite dowels with pry-out failure. But so far, the internal mechanisms controlling the pry-out behavior in cracked concrete have not been clarified. Therefore, a experimental study\textsuperscript{10} on the shear behavior of composite dowels in cracked concrete has been performed. These tests were conducted in an innovative test setup allowing to investigate the influence of crack spacing and crack width. The experimental results are summarized in Figure 1. Figure 1 (top, left) illustrates the influence of the crack spacing. A reduction of the crack spacing leads to a decrease of the average shear capacity. This decline in shear capacity with decreasing crack spacing (or increasing number of cracks) can be explained with the concrete pry-out cones’ dimensions. While, in uncracked concrete, the pry-out cone diameter correlates to six-fold the pry-out cone’s depth, transverse cracks yield a remarkable reduction of these dimensions (Figure 1, top, right). The cracks cut through the originally coherent pry-out cones and detach them into several smaller pry-out cone slices. The length of these pry-out cone segments correlates approximately the average crack spacing $s_r$ of the transverse cracks.

Furthermore, figure 1 (bottom, left) shows the influence of the crack width onto the pry-out capacity. Here, a degressive correlation between shear strength and crack width can be found, which tends towards a horizontal asymptote for very large crack widths. Figure 1 (bottom, right) shows the pry-out cones under different transverse crack widths. While for small crack widths large pry-out cones occurred, the pry-out cone surface reduced with increasing crack width. This is due to the effectivity of the aggregate interlock mechanism, that controls the transfer of shear stresses across the crack. For small crack widths, the flanks of the transverse crack feature an effective interlocking so that the blowout force, acting orthogonal to the shear force, is almost completely transferred across the transverse crack. Thereby, large and coherent pry-out cones occur and high shear capacities are reached. In tests with larger crack widths, the effectiveness of the aggregate interlock is reduced so that the transfer of shear stresses across the crack is affected. Due to the reduced shear transfer, the pry-out cones cannot bridge the discontinuity. They are cut off by the cracks and have significantly reduced dimensions (Figure 1, bottom, right).
3 Development of engineering models

The experimental investigations (chapter 2) show, that the pry-out behaviour of composite dowels in transversely cracked slabs is generally based on the same structural mechanisms like in uncracked concrete. Regarding that transverse concrete cracks appear as discrete discontinuities of the pry-out cone, in which the shear transfer is limited to aggregate interlock, the known pry-out-models can be extended for the use in cracked concrete\textsuperscript{11}.

3.1 Pry-out model for uncracked concrete

The pry-out-model for composite dowels in uncracked concrete from\textsuperscript{6} is based on models describing the blow-out-failure of stud connectors near edges\textsuperscript{12}. Figure 2 illustrates the structural behavior of composite dowels with pry-out failure. The applied shear force is transferred from the steel dowel onto the concrete dowel through contact compression stresses. Immediately in front of the steel dowel a highly stressed concrete wedge arises. This concrete wedge is confined by the surrounding concrete leading to the development of a multi-axial stress state. The confined concrete wedge causes a blowout force, acting in the direction orthogonal to the applied shear force, towards the smallest concrete cover (cf. Figure 2). The shear capacity $P_{po}$ can be expressed through the maximum blowout force $T_{po}$ multiplied with the factor $1/\eta$, considering the angle of diversion between shear and blow-out direction (1).

$$P_{po} = \frac{1}{\eta} \cdot T_{po}$$ (1)
The factor \( (1/ \eta) \) strongly depends on the concrete strength. For high strength concretes (HSC), the increase in strength caused by the multi-axial compressive stress is minor than for normal-strength concrete (NSC). At the same time, the occurring transverse strains of HSC increase significantly compared to NSC. In consequence, for high-strength and ultrahigh-strength concretes the values of \( (1/\eta) \) decrease. According to HEINEMEYER, the influence of the concrete strength is considered through equation (2).

\[
\eta = 0.4 - 0.001 \cdot f_{ck} \tag{2}
\]

Improved confinement of the concrete wedge can be achieved by inserting extra reinforcement around the composite dowel. The use of additional transverse reinforcement yields an increase in pry-out capacity. The model considers the effect of transverse reinforcement through the increase factor \( (1 + \rho_D, i) \) with \( \rho_D, i \) representing the degree of transverse reinforcement (3):

\[
P_{po} = \frac{1}{\eta} \cdot (1 + \rho_D, i) \cdot T_{po} = \frac{1}{\eta} \cdot (1 + \frac{A_{s,f} \cdot E_s}{A_{D,i} \cdot E_c}) \cdot T_{po} \tag{3}
\]

with \( A_{D,i} = h_c \cdot e_x \) and \( A_{s,f} = A_b + A_t \)

The pry-out failure occurs when the tensile strength of the concrete is exceeded in the surface of the pry out cone. Therefore, the bearable blowout force \( T_{po} \) depends on the tensile strength of the concrete and the size of the surface of the break-out cone. The concrete tensile strength can be assumed proportional to the square root of the compression strength \( k_1 \cdot \sqrt{f_{ck}} \). The surface of the break-out cone depends on the square of the cone height \( h_{po} \). For dowel geometries given in the technical approval, \( h_{po} \) can be calculated with (4):

\[
h_{po} = \min(c_t + 0.07 \cdot e_x; c_b + 0.13 \cdot e_x) \tag{4}
\]

When the pry-out cone height \( h_{po} \) is very small, the tensile stresses reach the tensile strength of the concrete almost completely in the break-out cone surface. Hence, the bearable blowout force \( T_{po} \) is proportional to the product \( k_1 \cdot k_2 \cdot \sqrt{f_{ck}} \cdot h_{po}^2 \). In contrast, for large embedment depths and cone heights \( h_{po} \) a significant size effect occurs, as the tensile strength is only reached at the crack tip and
flattens along the cracked surface. This size effect leads to a dependence of $h_{po}^{1.5}$ instead of $h_{po}^{2}$.

Combining the constant factors ($k = k_1 k_2$) equation (5) results for the definition of $T_{po}$:

$$T_{po} = k \cdot \sqrt{f_{ck}} \cdot h_{po}^{1.5} \quad (5)$$

$$P_{po} = \frac{1}{\eta} \cdot (1 + \rho_{D,i}) \cdot k \cdot \sqrt{f_{ck}} \cdot h_{po}^{1.5} \quad (6)$$

Equation (6) describes the pry-out shear capacity of a single shear connector. Since composite dowels are usually arranged in a row, effects from overlapping pry-out cones need to be considered, in addition. The interaction of adjoining pry-out cones yields a reduction of the pry-out capacity. To account for overlapping pry-out cones, the factors $\chi_x$ and $\chi_y$ were introduced. $\chi_x$ specifies the overlapping in longitudinal- and $\chi_y$ in transverse direction ($\chi_y$ only needs to be considered, in cases where several composite dowel bars are installed parallel to each other; otherwise $\chi_y$ is 1.0). The approach for $\chi_x$ is given in equations (7, a) and (7, b). For tests on specimen with only few composite dowels in a row, equation (7, b) is recommended allowing to consider the fact, that in a row of $n$ connectors only $n-1$ pry-out cones are affected by overlapping effects. For the design of composite beams with numerous shear connectors, their actual number is of minor importance and may be neglected (7, a).

$$\chi_x = \frac{e_x}{4.5 \cdot h_{po}} \quad (a) \quad \text{and} \quad \chi_x = \frac{1}{n} + \frac{e_x}{4.5 \cdot h_{po}} \cdot \left(1 - \frac{1}{n}\right) \quad (b) \quad (7)$$

The complete engineering model for the determination of the shear strength of composite dowels with pry-out failure is given in equation (8). Here, the factor $\chi_y$ is 1.0 for composite beams with only one single row of composite dowels:

$$P_{po} = \frac{1}{\eta} \cdot \chi_x \cdot \chi_y \cdot (1 + \rho_{D,i}) \cdot k \cdot \sqrt{f_{ck}} \cdot h_{po}^{1.5} \quad (8)$$

### 3.2 Pry-out model for cracked concrete

In the following, the pry-out-model (3.1) is extended to the use in transversely cracked concrete. Figure 3 illustrates the idealized pry-out mechanism and the interaction with transverse cracks. The cracks have a mutual spacing $s_r$ and divide the break-out cone with the length $l_{cone}$ in $n$ segments. The length $l_{cone}$ is equivalent to six-fold the break-out cone’s height$^{12}$. In figure 3 the concrete cone segments are indicated with capital letters A, B, C and the cracks are numbered from $i = 1$ to $i = n - 1$. The number of pry-out cone segments is determined by equation (9). It is based on the worst case, that the transverse cracks and the pry-out cone edges are shifted against each other. Equation (9) is to be rounded to natural numbers.
Figure 3: Pry-out-model for the transversely cracked concrete: Longitudinal cut through the pry-out cone (left), section through the transverse crack 2 (right).

\[ n = \frac{l_{cone}}{s_r} + 1 = \frac{6 \cdot h_{po}}{s_r} + 1 \tag{9} \]

The blowout force of the composite dowel, acting orthogonal to the shear force, is initially concentrated in segment A, touching the steel dowel immediately. Through continuously increasing the shear force, there comes a moment when the ultimate blowout force \( t_{po,A} \) of segment A is exceeded. Based on the fundamental assumption of the CC-method\(^\text{13} \), that all \( n \) equally sized segments have the same blow-out capacity, \( t_{po} \) can be calculated through equation (10).

\[ t_{po,A} = t_{po,B} = \ldots = t_{po,n} = \frac{T_{po}}{n} = \frac{k \cdot \sqrt{f_{ck}} \cdot h_{po}^{1.5}}{n} \tag{10} \]

To allow for further shear force increase after reaching \( t_{po,A} \), the adjoining pry-out cone segment B has to get involved in the shear carrying mechanism. Therefore, a transfer of shear stresses from A to B is necessary. The flanks of the transverse crack \( i = 1 \) allow for shear stress transfer due to aggregate interlock. The effectiveness of the aggregate interlock mechanism \( \tau_{cr} \) depends on numerous parameters, for example the crack width and the aggregate size of the concrete. The bearable shear stresses \( \tau_{cr} \) are described mathematically in equation (11) after \textsc{vecchio & collins}\(^\text{14} \), with \( D_{max} \) representing diameter of maximum aggregate size.

\[ \tau_{RR} = 0.18 \cdot \tau_{max} \cdot \frac{\sqrt{f_{cm}}}{0.31 + 24 \cdot w_r / (D_{max} + 16)} \tag{11} \]

The shear stress transfer \( \tau_{cr} \) takes place in the crack surface of crack \( i = 1 \), lying between the pry-out cone segments A and B (figure 3, right). The maximum blowout force \( t_{cr,i=1} \), that can be transferred across the crack \( i = 1 \) onto segment B, can be determined through integrating the shear stresses \( \tau_{cr} \) in the crack surface. Under the assumption of a rectangular crack surface with the height \( h_{i=1} \) and the width \( b_{i=1} \) equation (12) results for \( t_{cr,i=1} \):

\[ t_{cr,i=1} = t_{cr,B} = \tau_{cr} \cdot h_{i=1} \cdot b_{i=1} \tag{12} \]
The pry-out cone’s height $h_i$ and width $b_i$ reduce with increasing spacing between crack and shear connector. In consequence, the dimensions of the crack surface, that is activated during shear stress transfer, depend on the location of the crack. The crack index $i$ is an indication for the distance between crack and shear connector and identifies explicitly the location of each transverse crack. Therefore, $i$ is used in equation (13) to describe the location. The height of the crack area $h_i$ decreases linear from the edge of the shear connector (here: $h_i$ is the maximum height of the pry-out cone $h_{i=1} = h_{po}$) to the edge of the pry-out cone ($h_{i=\infty} = 0$). The width of the crack surface reduces linear from the edge of the shear connector ($b_{i=1} = 6 \cdot h_{po}$) to $b_{i=\infty} = h_{po}$ at the pry-out cone edge (figure 3, right).

$$h_i = h_{po} \cdot \frac{n - i}{n - 1}$$
$$b_i = h_{po} \cdot \left(1 + 5 \cdot \frac{n - i}{n - 1}\right)$$

With equations (10) and (12), two different criteria are available to determine the maximum blowout force of segment B. Whereby equation (10) describes the blowout force, for which pry-out of the segment would be governing, on the contrary, equation (12) describes the force, for which aggregate interlock between adjoining segments would fail. Apparently, the force $t_{max,B}$, contributed from segment B to the overall shear capacity, is defined as the smaller value of $t_{po,B}$ and $t_{cr,B}$:

$$t_{max,B} = \min\left\{t_{po,B}, t_{cr,B}\right\} = \min\left\{\frac{k \cdot \sqrt{f_{ck}} \cdot h_{po}^{1.5}}{\tau_{cr} \cdot h_{i=1} \cdot b_{i=1}}, n\right\}$$

Figure 3 shows a case where the aggregate interlock between segments A and B stays intact. Due to the adequate interlocking of the crack flanks the blowout force $t_{cr,B}$ transferred to segment B is higher than the corresponding pry-out capacity $t_{po,B}$ of the segment. Therefore, the crack between segments A and B is effectively bridged and a continuous pry-out cone for segment A and B develops.

However, for crack $i=2$ between segments B and C the aggregate interlock criterion is governing, since the activated crack surface is comparatively small. Obviously, the blowout force $t_{cr,C}$, transferred by aggregate interlock, is smaller than the pry-out capacity $t_{po,C}$ of segment C. Therefore, the coherent pry-out cone of segments A and B is cut at the transverse crack $i=2$. In the following, crack $i=2$ is called the “critical crack”. During the pry-out process the force $t_{cr,C}$, caused by the relative displacement of the crack flanks in $i=2$, is activated. Thereby it contributes to the overall blowout force $T_{R,po}$:

$$T_{R,po} = t_{po,A} + t_{max,B} + t_{max,C} = t_{po,A} + \sum_{i=1}^{n-1} t_{max,i}$$

Using equation (8), (15) yields equation (16) for the shear capacity in cracked concrete:
\[ P_{po} = \frac{1}{\eta} \cdot (1 + \rho_{D,i}) \cdot \chi_x \cdot \chi_y \cdot T_{R,po} \]

\[ = \frac{1}{\eta} \cdot (1 + \rho_{D,i}) \cdot \chi_x \cdot \chi_y \cdot \left[ k \cdot \sqrt{f_{ck}} \cdot h_{po}^{1.5} + \sum_{i=1}^{n-1} \min \left( \frac{k \cdot \sqrt{f_{ck}} \cdot h_{po}^{1.5}}{\tau_{cr} \cdot h_i \cdot b_i} \right) \right] \]  

Equation (16) can be simplified and transferred into equation (17), which is identical with the well-known pry-out equation (8), extended by the factor \( \psi_{crack} \) to consider the reduction of shear capacity due to transverse cracks:

\[ P_{po} = k \cdot \frac{1}{\eta} \cdot (1 + \rho_{D,i}) \cdot \chi_x \cdot \chi_y \cdot \sqrt{f_{ck}} \cdot h_{po}^{1.5} \cdot \psi_{crack} \]

\[ \psi_{crack} = \left[ \frac{1}{n} + \sum_{i=1}^{n-1} \min \left( \frac{1}{n} \cdot \frac{\tau_{cr} \cdot h_i \cdot b_i}{k \cdot \sqrt{f_{ck}} \cdot h_{po}^{1.5}} \right) \right] \]

Using equations (15) till (17), an abort criterion for the algorithm has to be considered: Only the pry-out capacities of the segments lying closer to the composite dowel than the critical crack shall be summed up. All segments having larger distances from the composite dowel than the critical crack do not contribute to the pry-out capacity. Therefore, the summation in (15) till (17) shall be stopped after \( i \) exceeds the number of the critical crack.

4 Comparison of the consistent model with the test results

With the novel approach (chapter 3.2), the pry-out-model for uncracked concrete gets extended by a simple mechanical model considering aggregate interlock. To validate the quality of the extended model a recalculation of the tests\(^{10} \) was performed. In this preliminary evaluation, the coefficient \( k \) in equation (17) was set to \( k = 35 \), so that for the uncracked reference test S0/SR0 an exact accordance between the model and the test results. Through this, all inaccuracies of the model, which are not related to the crack modelling (aggregate interlock), are excluded and do not falsify the examination.

Figure 4 illustrates the comparison of the test results and the model. While the shear capacity calculated with the model is a continuous function of the crack width (Figure 4, right), a non-continuous relation between the crack spacing and the shear capacity is found (Figure 4, left). The stepwise descending function is due to equation (9) requiring to round off the number of segments to natural numbers. Through this, equation (9) generates a constant number of segments for every crack spacing \( s_r \) within a specific range. For example, (9) yields \( n = 2 \) pry-out cone segments for each crack spacing between 16 cm and 50 cm. In consequence, the calculated shear capacity is constant for each crack spacing within this range. All in all, the model allows to simulate both, the impact of the crack spacing (Figure 4, left) and the influence of the crack width (Figure 4, right) accurately.
5 Experimental database for composite dowels with pry-out failure – Calibration of the pry-out-model

The mechanically based pry-out-model contains the coefficient $k$, in which the conversion of $\sqrt{f_{ck}}$ to the tensile strength and of $h_{po}^2$ to the pry-out cone surface are combined. Generally, the determination of $k$ is an empirical process, using test results. As the determination of this coefficient has remarkable influence on the quality of the model, a large data basis is required. In this paper, data is taken from a comprehensive composite-dowel-data base, which contains collected data from several researchers\textsuperscript{4,5,8,9,10,15}. The data base\textsuperscript{11} covers 87 tests (52 in uncracked concrete, 36 in cracked concrete). The statistical evaluation based on these tests leads to a coefficient $k$ of 41 (18):

$$
P_{po} = 41 \cdot \frac{1}{\eta} \cdot (1 + \rho_{D,i}) \cdot X_x \cdot X_y \cdot \sqrt{f_{ck}} \cdot h_{po}^{1.5} \cdot \psi_{crack}$$

$$
\psi_{crack} = \left[ \frac{1}{n} + \sum_{i=1}^{n-1} \min \left( \frac{1}{n} \cdot \tau_{cr} \cdot h_1 \cdot b_i \right) \right]$$

In Figure 5 the ratio of experimental and theoretical shear capacity is plotted over the input variables of the model for all tests. The uniform pry-out-model considers all input parameters (concrete strength, ratio of reinforcement, pry-out cone height as well as the crack width and crack spacing) correctly and without substantial trends and divergences. As the own approach is an extension of the pry-out-model given in the technical approval\textsuperscript{6}, the statistical evaluations of both models are contrasted in table 1. If only the tests with uncracked concrete are considered (table 1, row I), both models produce (nearly) identical results with a very good accuracy. The own approach provides a coefficient of variation (COV) of 18% and a mean value calibrated to 1.0. With a mean value of 0.98 and a COV of 18% the model of the technical approval\textsuperscript{6} yields practically identical results. The minimal discrepancy leads back to the slightly different prefactors $k$. However, if the test results in cracked concrete are considered in the evaluation (Table 1, row II) remarkable differences become apparent. As the technical approval\textsuperscript{6} does not consider the impact of concrete cracking, the model overestimates the shear capacities of dowels in cracked concrete by in average 35% with a COV of 29%.
Figure 5: Ratio of experimental and theoretical shear capacity plotted over the influencing parameters

Table 1: Comparison of the pry-out model from\textsuperscript{12} with the own approach

<table>
<thead>
<tr>
<th>Test series</th>
<th>all</th>
<th>4</th>
<th>5</th>
<th>8</th>
<th>9</th>
<th>15</th>
<th>10 Cracked concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of tests</td>
<td>88</td>
<td>2</td>
<td>11</td>
<td>9</td>
<td>21</td>
<td>9</td>
<td>36</td>
</tr>
<tr>
<td>$X_m = P_{\text{test}}/P_{\text{theo}}$</td>
<td>0.98</td>
<td>1.27</td>
<td>0.97</td>
<td>0.95</td>
<td>0.99</td>
<td>0.95</td>
<td>-</td>
</tr>
<tr>
<td>$V$</td>
<td>0.18</td>
<td>0.03</td>
<td>0.13</td>
<td>0.18</td>
<td>0.20</td>
<td>0.16</td>
<td>-</td>
</tr>
<tr>
<td>I</td>
<td>6</td>
<td>$X_m$</td>
<td>1.00</td>
<td>1.32</td>
<td>1.00</td>
<td>1.01</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V$</td>
<td>0.18</td>
<td>0.03</td>
<td>0.15</td>
<td>0.17</td>
<td>0.20</td>
</tr>
<tr>
<td>equation (18)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>6</td>
<td>$X_m$</td>
<td>0.82</td>
<td>1.27</td>
<td>0.97</td>
<td>0.95</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$V$</td>
<td>0.31</td>
<td>0.03</td>
<td>0.13</td>
<td>0.18</td>
<td>0.20</td>
</tr>
<tr>
<td>equation (18)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Caption:
- [4]
- [5]
- [8],[10]
- [9]
- [15]
Regarding the whole database, the consideration of the 36 tests with cracked concrete yields a reduction of the mean value to 0.82 and a COV of 31%. In contrast, the own approach describes the results of the test series in cracked concrete with very good accordance (mean value of 0.97, variation coefficient 15%) and leads to a mean value of 0.98 and a COV of 16% for the whole database.

6 Summary and Outlook

In the present paper, an existing pry-out model for composite dowels was extended by the influence of transverse concrete cracking. The basic idea of the developed approach is to assess the shear capacity in comparing the force, which can be transferred across the critical transverse crack by aggregate interlock with the pry-out force of the corresponding pry-out cone segment. The novel approach allows to recalculate existing shear tests of composite dowels in cracked concrete with good agreement. The developed pry-out model was validated using a comprehensive database of tests in cracked and uncracked concrete. The model was transferred to the design load level considering the relevant safety and reliability aspects\textsuperscript{11}.

The developed modeling approach for the interaction of composite dowels and transverse cracks can also be adopted and applied to other fields of application such as anchoring of reinforcement and fastening technology, where concrete cracking also leads to a reduction in load bearing capacity, as shown by ELIGEHAUSEN\textsuperscript{12,16,17,18}.

References


STEEL-TO-CONCRETE JOINTS WITH LARGE ANCHOR PLATES UNDER NORMAL AND CONSTRAINING FORCES

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ABSTRACT

In industrial and plant buildings advantages of concrete construction are combined with those of a steel or steel-concrete composite construction. Rising demands on this mixed building technology to achieve an optimal utilisation of the materials also need an optimisation of the connection configuration in order to apply higher loads of steel or steel-concrete composite construction. Due to current normative codes the number of fasteners is limited to a maximum of nine per anchor plate. The design of steel-to-concrete joints by the codes is often conservative.

1 Introduction

The research project "Large anchor plates with headed studs for highly stressed structures in industrial and plant constructions" deals with the investigation on the load-bearing behaviour of large anchor plates with more fasteners than currently permitted by codes under tensile-, shear- and constraining forces. In collaboration with the University of Stuttgart, University of Kaiserslautern investigated the behaviour of large thick and thin anchor plates under tension loads and constraining forces. Parameters like the thickness of the anchor plates, the effective embedded depth of the headed studs, the ratio of a supplementary reinforcement and the state of the concrete were detected as major components for a suitable design model and have been tested in static and short-time relaxation experiments.

This paper describes tension and constraining forces on large anchor plates with headed studs and gives an overview about the developed component models based on the component method of EN 1993-1-8¹ as well as the load capacity model of EN 1992-4³.

2 State-of-the-art

2.1 Normative Framework

There are different codes for connections between steel and concrete. Typical connections are realised by anchor plates with headed studs. The headed studs transfer occurring shear and tension loads as mechanical fasteners. The fasteners are thereby calculated by normative documents as listed: concrete elements by EN 1992-4³, steel components by EN 1993-1-8¹ and steel-concrete composite elements by EN 1994-1-1⁴. In addition, there are different European Technical Approvals for the particular product. For the steel and steel concrete construction the model of the equivalent T-Stub
was implemented by the codes mentioned above. The determination of the load bearing and the deformtion behaviour is already possible. The basis of the fastener calculation represents the concrete capacity method\textsuperscript{5,7} (CC-Method) with the calculation of the load bearing behaviour of all concrete elements. This method is only valid for the load capacity and not for the deformation behaviour. For the calculation of the deformation new design models\textsuperscript{6} have been developed.

### 2.2 Concept for the fastening technology

In the fastening technology, the load capacity is calculated by single components. The minimum of these loads is thereby defining the total capacity of a connection\textsuperscript{7}. At this juncture, the capacity under a tensile load is calculated for the following single components: Cone-shaped concrete failure, extraction of fasteners (Pull-out / Pull-through failure), split of the concrete as well as steel failure of fasteners. Furthermore, in the range of edges, the local blow-out failure and the concrete edge failure can occur. In EN 1992-4\textsuperscript{3} the load capacities of shear and tension are differentiated and determined separately. For the consideration of the total connection, the interaction conditions are implemented. Analogous to the CC-Method the smallest load capacity of the single components is defining the total load capacity of a connection configuration. Parallel to the CC-Method single headed studs as well as groups in the range of edges and far away from edges are differentiated.

### 2.3 Technical principals of the component method

EN 1998-1-8\textsuperscript{1} and EN 1994-1-1\textsuperscript{4} give a design model using the equivalent T-Stub, which provides the opportunities of the partitioning of a connection in the single components and the particular determination of load capacities as well as stiffness’s in steel and steel-concrete composite construction. With the help of a design model the classification of joints can be determined and a construction therefore can be calculated more economically. Here it is distinguished between three calculation methods: normally pinned, semi-rigid and rigid. The connection classification occurs also in three fields: moment resistance, rotational stiffness and rotation capacity.

#### 2.3.1 Model of the equivalent T-Stub

With the Model of the equivalent T-stub three different failure mechanisms can be described. Mode 1: Complete yielding of the flange, Mode 2: Bolt failure with yielding of the flange and Mode 3: Bolt failure. Former research projects\textsuperscript{8,9,10} have improved the model of the T-stub and extended it to more than two bolts per row.

#### 2.3.2 Consideration of a supplementary reinforcement

The consideration of supplementary hanger reinforcement in the field of tensional loading was evaluated in further research projects\textsuperscript{6}. There an increase of load capacity by the model of parallel use of the load capacities of concrete and the supplementary hanger reinforcement was developed and verified. A too short anchoring depth, however, causes another failure mechanism, the small concrete strut. Here, the resulting strut of the headed stud places itself steeper in the direction of the stirrups of the hanger reinforcement. From the stirrups, a small concrete cone occurs. Based on this model, the influence of the supplementary hanger reinforcement was evaluated for an enclosing surface reinforcement\textsuperscript{11}. The results of these studies have been considered for the development of the analytical model.
2.3.3 Consideration of the surface reinforcement in the area of large anchor plates
The CC-method shows that a surface reinforcement leads to a reduction of the maximum load of the concrete capacity. This reduction is described by a factor $\psi_{re,N}$ and therefore considers the center distance of the reinforcement as well as the effective bonding depth of the fasteners. This model has been developed for small anchor plates and is verified in a variety of tests. By the formation of bigger concrete cones there will be a support of the deforming surface reinforcement in the Post-Peak behaviour. A model of deviation for impact loads was described by Schlüter\textsuperscript{12}.

2.4 Constraining forces in the area of anchor plates
Constraining forces as a result of time-dependent effects have been investigated on small anchor plates. There have been applied thermal loads, hence it comes to an extension of the anchor plate. It can be summarized, that the thermal loads investigation\textsuperscript{13} did not cause a significant change of the fastener’s stress. The constraints trough a rapid heating had solely effects of the stress state within the steel plate. These results were considered for the conception of the test specimen.

3 Experimental Investigations under normal forces

3.1 General concept
The experimental investigation deals with the tension area that forms in a 4x4 anchor arrangement and is oriented on tests of former research projects\textsuperscript{14}. The tension zone shows a 4x2 anchor arrangement. Based on the finding that a hanger reinforcement causes an increasing load capacity different grades of supplementary reinforcement were arranged. For the validation of an analytical model by the test results, solely the boundary conditions have been evaluated experimentally. Thereby initially a thick anchor plates with a small embedded depth of the headed studs achieved a concrete failure and the thin anchor plates with a large embedded depth achieved a steel failure. Beside the load capacity the strains and deformations on relevant areas have been measured.

3.2 Experimental testing under normal forces
The following Table 1 describes the specimen with the major parameters which were detected, based on the current codes and regulations.

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>Thickness of the anchor plate [mm]</th>
<th>Headed stud</th>
<th>State of the concrete</th>
<th>Hanger reinforcement per headed stud</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1-N1</td>
<td>40</td>
<td>SD16/100</td>
<td>cracked</td>
<td>1x Ø 8mm</td>
</tr>
<tr>
<td>B1-N2</td>
<td>40</td>
<td>SD16/100</td>
<td>cracked</td>
<td>1x Ø 8mm</td>
</tr>
<tr>
<td>B2-N</td>
<td>40</td>
<td>SD16/100</td>
<td>cracked</td>
<td>2x Ø 8mm</td>
</tr>
<tr>
<td>B3-N</td>
<td>40</td>
<td>SD16/100</td>
<td>cracked</td>
<td>2x Ø 8mm</td>
</tr>
<tr>
<td>R5-1N</td>
<td>15</td>
<td>SD16/250</td>
<td>uncracked</td>
<td>2x Ø 8mm</td>
</tr>
<tr>
<td>R5-2N</td>
<td>15</td>
<td>SD16/250</td>
<td>uncracked</td>
<td>2x Ø 8mm</td>
</tr>
<tr>
<td>R5-3N</td>
<td>15</td>
<td>SD16/250</td>
<td>cracked</td>
<td>2x Ø 8mm</td>
</tr>
</tbody>
</table>

Due to the anchor plate thickness and the achiving failure mechanism the tests are divided into two series. Both test series where conducted in uncracked and cracked state of the concrete. The
geometry of the specimen as well as the arrangement and grid of the surface reinforcement was similar in all the tests, see Figure 1.

![Figure 1: Geometry of the specimen with the supplementary hanger reinforcement (left)
Specimen in the test stand (right)](image)

### 3.3 Experimental results under normal forces

The experimental results have shown, that anchor plates with a small embedded depth of the headed stud have a small effectiveness of the hanger reinforcement. An induced crack in the axis however, affects the load capacity significantly. Here the factor 0.75 through an induced crack, should be used.

![Figure 2: Load deforming diagram of the experimental testing under normal forces
Left: B-N series – Right: R5-N series](image)

Thereby the reference tests B1-N solely showed the concrete failure by a cone shaped failure respectively a small compression strut failure. Test series R5-N showed an early splitting crack occurrence starting from the head of the headed stud orthogonally to the edges of the specimen. As a result, there wasn’t a significant difference in the load capacities of the cracked and the uncracked condition by series R5-N. Both of the tests R5-1N and R5-3N showed in addition plastic deformations in the anchor plate. Here a T-Stub has formed. Through the variation of the anchor plate width in the test R5-2N and the bigger distance of the prying forces to the headed studs the anchor plate stood in the elastic range.
In Figure 4 primary and secondary concrete failure cone can be seen clearly. The primary failure cone, braces itself on the grid of the surface reinforcement. It leads to a secondary failure cone caused by the small concrete struts. As a result of the surface reinforcement deflection, through the bracing of the primary as well as the secondary failure cone, a successively blast of the concrete cover occurs. The investigations have shown a typical behaviour of the anchor plates with headed studs. The failure mechanism of the limit cases has been confirmed. The results and the observations of both of the test series can be considered for a design model for a moment load and therein a separate approach of a 4x4 stud arrangement.

4 Numerical Investigations under normal forces

4.1 General concept

The modelling of the tests under normal forces was performed with the Software AutoCAD (Autodesk) and was imported to Abaqus (Simulia). The specimens were optimized for numerical investigation. The surface reinforcement and the supplementary reinforcement were modelled as truss elements. This step is useful for a continuous meshing of the concrete part which is important for the damage model of the concrete.

4.1.1 Materials

As for the concrete model the non-linear Concrete Damaged Plasticity Model was used. This has significant advantages for the modeling of concrete in a tension and pressure range. According to this model $\varepsilon_{\text{tip}}$ and $\varepsilon_{\text{cpl}}$ are implemented$^{15,16}$, which describe the beginning of a tension or pressure
damage. By damage theory, the stiffness decreases by an ever-larger elongation. This concrete model is basically based on a Drucker-Prager model approach. As for steel, a bilinear approach is employed. The occurring elongations could be modelled sufficiently accurate by this approach.

4.2 FE-model of the experimental investigations under normal forces

The FE-model and the experimental investigations have good corresponding results and can be compared in the figure below. The test specimen shows the failure mechanism of a thick anchor plate with short headed studs in an uncracked state of the concrete.

Figure 5: Comparison of the tensional damage of the experimental and numerical investigations

Figure 5 describes a tension failure of the concrete, green approx. 40% damaged and red approx. 80% damaged. The output of the damage showed that a crack forms from the headed stud and a cone-shaped concrete failure develops. With the activation of the hanger reinforcement and a bracing of the resulting breakout cone by the surface reinforcement, the compression strut place itself steeper until the peak of the rebar of the hanger reinforcement is reached. These failure mechanisms of the experimental investigations, have been confirmed by this numerical model. Additional numeric test results can be taken from the final report18 as well as from the test report19 of the research project.

4.3 Verification of the FE-model under normal forces

Due to the described implementation of the model of tensile damage, the splitting-cracks as well as the concrete cone have been simulated. Figure 6 shows a comparison of the load capacity out of the experimental test to the numerical simulations. Beyond all tests, a good correspondence of the load capacity as well as the failure mechanisms has been achieved. The results of the load capacity of the numerical simulation ($F_{u,FEM}$) and the tests ($F_{u,Exp}$) indicate a standard deviation of 3.56%.

<table>
<thead>
<tr>
<th>Nr.</th>
<th>Test specimen</th>
<th>Load capacity test series</th>
<th>Load capacity FEM</th>
<th>$F_{u,FEM}/F_{u,Exp}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B1-N1</td>
<td>222</td>
<td>232</td>
<td>1.05</td>
</tr>
<tr>
<td>2</td>
<td>B1-N2</td>
<td>222</td>
<td>232</td>
<td>1.05</td>
</tr>
<tr>
<td>3</td>
<td>B2-N</td>
<td>221</td>
<td>213</td>
<td>0.96</td>
</tr>
<tr>
<td>4</td>
<td>B3-N</td>
<td>294</td>
<td>283</td>
<td>0.96</td>
</tr>
<tr>
<td>5</td>
<td>R5-1N</td>
<td>627</td>
<td>576</td>
<td>0.92</td>
</tr>
<tr>
<td>6</td>
<td>R5-2N</td>
<td>571</td>
<td>572</td>
<td>1.00</td>
</tr>
<tr>
<td>7</td>
<td>R5-3N</td>
<td>612</td>
<td>576</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>Average:</td>
<td></td>
<td></td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>Standard deviation:</td>
<td></td>
<td></td>
<td>3.56 %</td>
</tr>
</tbody>
</table>

Figure 6: Comparison of the test specimen and numerical load capacity
Based on these results, further numeric investigations have been conducted. There, parameters such as the concrete compressive strength, the embedding depth of the headed studs, the hanger reinforcement ratio and the ratio of the surface reinforcement were varied. The results of these simulations are shown in comparison to the analytical model in Figure 7.

5 Component model for large anchor plates under normal forces

5.1 General

In the following, an analytical model for connections with large anchor plates will be proposed. The components describe the headed studs under a tensile load with the consideration of an increase of load capacity by hanger reinforcement. The analytical model has been verified by the implemented tests as well as the numerical simulations, and can be obtained from Figure 7.

Figure 7: Comparison of the analytical model under tensional forces and the experimental investigations (left) and further parameters of the FE-Model (right)

- P1: Variation of concrete compressive strength and embedded depth of the headed studs
- P2: Variation of the supplementary hanger reinforcement
- P3: Variation of the surface reinforcement

The complete analytical model under shear and tensile forces is described in article “Steel-to-Concrete Joints with Large Anchor Plates”. In the following chapter the component model for large anchor plates under normal forces is described. As stated at the beginning, this model is based on the component method of EN 1993-1-81 and the design model of EN 1992-43. Thereby the description of the single components under tensile forces of former research projects are extended.

5.2 Load-bearing capacity of an anchored group under normal forces

5.2.1 General

The spring model in Figure 8 describes the load capacity of a connection as well as the load capacity of the post peak behavior. In the post peak behavior, the additional influence of the surface reinforcement due to a bracing of the concrete cone is implemented. This describes consequently the deformation behaviour. The single components of the model are based on the concrete components of the CC-method of the fastener technology as well as on the

Figure 8: Spring model for the tensional components. Left: Pre-Peak (load capacity); Right: Post-Peak behaviour with the new description of the surface reinforcement $N_{Rd,sre}$
extension of the T-stub model. Table 2 shows the calculation of load capacities of the single components without consideration of partial safety factors.

Table 2: Load capacity of the single components under tensional loads for headed studs

<table>
<thead>
<tr>
<th>Failure Mechanism</th>
<th>Load capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel failure</td>
<td>$N_{RK,S} = A_s \cdot f_{u,S}$</td>
</tr>
<tr>
<td>Pull-Out / Pull-through</td>
<td>$N_{RK,P} = 10,5 \cdot A_h \cdot f_{c,K}$</td>
</tr>
<tr>
<td>Concrete failure of anchored groups</td>
<td>$N_{RK,C} = N_{RK,S}^0 \cdot \frac{A_s}{A_h} \cdot \psi_{ST,N} \cdot \psi_{f,E,N} \cdot \psi_{EC,N}$</td>
</tr>
</tbody>
</table>

- $N_{RK,S}$ characteristic resistance of a single headed stud
- $N_{RK,P}$ yield strength of the hanger reinforcement
- $N_{RK,C}$ yield strength of the hanger reinforcement
- $N_{RK,L}$ anchorage length
- $A_s$ cross-section area of the shaft
- $A_h$ bond strength of headed studs
- $f_{u,S}$ bond strength according to EC2
- $f_{c,K}$ bond strength of headed studs
- $f_{y,K}$ yield strength of the hanger reinforcement
- $f_{y,L}$ yield strength of the hanger reinforcement
- $f_{f}$ bond strength according to EC2
- $a$ factor according to EC2
- $\alpha_i$ influence factor of the hanger reinforcement
- $\psi_{f,E,N}$ support factor for not enclosing the surface reinforcement
- $\psi_{EC,N}$ support factor for enclosing the surface reinforcement
- $\psi_{f,E,N}$ support factor for not enclosing the surface reinforcement
- $\psi_{f,E,N}$ support factor for enclosing the surface reinforcement
- $\psi_{f,E,N}$ support factor for not enclosing the surface reinforcement
- $\psi_{f,E,N}$ support factor for enclosing the surface reinforcement
- $\psi_{f,E,N}$ support factor for not enclosing the surface reinforcement
- $\psi_{f,E,N}$ support factor for enclosing the surface reinforcement

Steel failure

$$N_{RK,RE} = n_{RE} \cdot A_{S,RE} \cdot f_{y,K}$$

Anchorage failure of the stirrups (in concrete cone and/or ground)

$$N_{RK,AN} = \sum n_{RE} \cdot \frac{\pi d_{s,RE} f_{BK}}{\alpha}$$

Concrete breakout and stirrups in tension

$$N_{u,RE} = N_{u,C} + \alpha \cdot \sum \min(N_{u,RE}, N_{u,AN})$$

Concrete breakout between supplementary hanger reinforcement

$$N_{u,CS} = \psi_{Supp} \cdot N_{u,C}$$

**Mode 1**

$$N_{T,1,RK} = i \cdot \frac{M_{pl,1,RK}}{m_x}$$

**Mode 2**

$$N_{T,2,RK} = \frac{2M_{pl,2,RK} + \pi S F_{L,RR}}{(m_x + n)}$$

**Mode 3**

$$N_{T,3,RK} = \sum F_{L,RR}$$
Table 3 describes the Post-Peak behaviour with the membrane effect of the surface reinforcement in case of an occurring concrete cone. Analogue to Table 2 the load capacity as a description of a successively blast of the concrete surface is shown. The component has solely an influence on the ductility of the connection in the post peak behaviour. The applied load on the grid of the surface reinforcement describes the peak load, for cone shaped concrete failure. (see Table 2). The extent of the concrete cone is important for the determination of the number of rods which cross the concrete cone. The length \( L \) describes a side of the idealized square-shaped concrete cone analogue to the CC-method.

### Table 3: Load capacity of the single components under tensile loads for headed studs

<table>
<thead>
<tr>
<th>Failure mechanism</th>
<th>Load capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Consideration of the surface reinforcement for the Post-Peak behaviour</td>
<td>( N_{Rk,ste} = 2 \cdot F_{M,Rd,i} \cdot n )</td>
</tr>
</tbody>
</table>

\[
A_c = (a_{sm} - d_{ave,m}) \cdot t \cdot \frac{2}{3}
\]

\[
d_{ave,m}
\]

\[
d_{ave,m}
\]

\[
t
\]

\[
F_{M,Rd,i} = A_c \cdot k \cdot f_{ct}
\]

and

\[
n = \frac{L}{a_{sm}}
\]

**Figure 9:** Idealistic concrete cone accordant to EC2 with crossing bars of the surface reinforcement

### Table 4: Analytical model for large anchor plates under tensile forces

<table>
<thead>
<tr>
<th>Mode</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Complete yielding of the flange according to equation (8)</td>
</tr>
<tr>
<td>2</td>
<td>For an anchor plate without a brace: ( N_{T,2,Rk} = 2m_{pl,platte,k}l_{eff,2} + \frac{N_{T,Rk}}{m_{x} + e} )</td>
</tr>
<tr>
<td>3</td>
<td>Minimum of the different failure mechanisms of Table 2</td>
</tr>
</tbody>
</table>

\[
N_{T,3,Rk} = \min \left\{ N_{Rk,i} \right\}
\]

**Figure 9:** Idealistic concrete cone accordant to EC2 with crossing bars of the surface reinforcement
5.2.2 Analytic model
The analytic model for the load capacity of the tension components of a connection as shown in Figure 8 could be described with the spring model. Based on EN 1993-1-8\(^1\) three main failure mechanisms, shown in Table 4, could occur.

6 Experimental Investigations under constraining forces

6.1 General concept
Based on the tests of former investigations\(^{13}\) no tests with thermal loads were performed. Constraining forces out of temperature result in an elongation of the anchor plate and therefore it comes to loads on fasteners. The same effect can occur through a constant load on the concrete, which leads to time-dependent deformation. The anchor plate with an arrangement of 6x2 headed studs was positioned sidewise on the concrete structure. Effects out of constraining forces due to the constant load, can be measured with strain gauges. The applied anchor plates had a thickness of \(t_p = 15\) mm and an embedded depth of \(h_{ef} = 242\) mm. Due to an expected large scatter of the tests, three similar specimens has been constructed.

6.2 Experimental procedure under constraining forces
The tests have been conducted in an uncracked state of the concrete. A constant size load of 600 kN has been applied for a duration of 8 hours.

The geometry of the specimen was \(h/w/d = 1500/1000/450\) mm. A concrete C20/25 was chosen. The hanger and a surface reinforcement was chosen analogue to the static tests under normal -and shear forces. Beside the strain gauges on all steel construction parts, there have been strain gauges at the height of the symmetrical axis on the concrete.

6.3 Experimental results under constraining forces
Due to the anchor plate at one side of the concrete structure, different strains were measured in the strain gauges on the concrete at both sides. These strains and the constant load are shown in Figure 11. A significant difference can be constituted for the side with and without anchor plate. The detailed description of the results of the strain gauges of the headed studs and rebars of the hanger reinforcement is given in the test report\(^{19}\) in addition to the research report\(^{18}\) of this project.

By holding the load constant at 600 kN, in the first hour there was a plateau forming in the strain curve. The decreasing strain of the steel components can be explained by the effect of short-time relaxation of the concrete at the beginning of the load application.
Summarizing the results, it can be said that an anchor plate with headed studs behaves like an external reinforcement. Thereby the stiffness on this area is increased. The tests have shown the behaviour, on which a design model could be developed. After all, the effect of the short time relaxation of the concrete and the related relaxation of the anchor plate can be seen as a positive effect on anchor plates.

7 Numerical Investigations under constraining forces

7.1 General concept
The analytical model is based on the experimental and numerical investigations. There was no failure in the test specimen, so the concrete material was implemented as linear elastic. In this case, the implemented Norton-Law in Abaqus can describe the effect of creeping for a testing duration of 8 hours as well as the long-time behaviour. Strain gauges at the horizontal symmetrical axis of the concrete, the anchor plate, the fasteners and on the rebars of the hanger reinforcement can be used for a verification of the FE-model.

7.2 FE-model of the experimental investigations under constraining forces
The numerical investigations show, that anchor plates show strains due to a vertical constraining load of the concrete. This strain is not constant over the length of the anchor plate and there are no significant bending effects over the height. These effects are shown in the symmetrical strain distribution of Figure 12.

7.3 Verification of the FE-model under constraining forces
The comparison of the numerical and experimental investigations shows that the short-time relaxation results slight by conservative outputs. For the calculation of large thin anchor plates with headed studs this effect of short-time relaxation is a positive effect. The different described levels of strain over the length of the anchor plate are defined by the rows of the headed studs. The strain of the concrete results in stresses for the fasteners. These loads are transferred to the anchor plate and were summed up so that the symmetrical axis shows the highest amount of strain.

Figure 11: Concrete strain and Load for a loading duration of 8 hours.

Figure 12: Comparison between the strain of the anchor plate and the strain of the numerical investigation.
8 Component method for large anchor plates under constraining forces

8.1 General
The developed spring model for large anchor plates under constraining forces is based on a rigid beam and the Bernoulli hypothesis. This model includes a consideration of time depending creeping and shrinkage effects of the concrete. The knowledge of the experimental and numerical investigations was used to build a truss model for several levels, which were described by the rows of headed studs and the anchor arrangement.

8.2 Creeping
Based on EN 1992-1-1\(^2\) the analytic model considered creeping as a modification of the Young’s modulus. When the elastic stress on the concrete, due to the vertical load, is below 0.4 \(f_{ck}\), linear creeping can be assumed using the tangent modulus. Increasing stresses have the consequence of non-linear creeping effects after EN 1992-1-1\(^2\).

\[
E_{c,eff} = \frac{E_{cm}}{1 + \varphi(\infty, t_0)}
\]

with:
- \(E_{cm}\): average modulus of elasticity of concrete
- \(\varphi(\infty, t_0)\): coefficient for creeping according to EN 1992-1-1

8.3 Shrinkage
For the consideration of shrinkage, a modified model of EN 1994-1-1\(^4\) has been developed. This model is shown below.

Shrinkage as a load independent effect of the concrete consists of autogenous and drying shrinkage.

\[
\varepsilon_{cs}(t, t_s) = \varepsilon_{cd}(t, t_s) + \varepsilon_{ca}(t)
\]

with:
- \(\varepsilon_{cd}(t, t_s)\): drying shrinkage
- \(\varepsilon_{ca}(t)\): autogenous shrinkage

The effect on anchor plates is that an additional normal force has to be considered acting on the composite cross section. This normal force can be described as:

\[
N_{cs} = E_{cm} \cdot A_c \cdot \varepsilon_{cs}(t, t_s)
\]

with:
- \(E_{cm}\): average modulus of elasticity of concrete
- \(A_c\): cross-section of the concrete
- \(\varepsilon_{cs}(t, t_s)\): total elongation of shrinkage

8.4 Analytical spring model for anchor plates under constraining forces
The analytical spring model for the different levels of headed studs is shown in Figure 14. The springs represent the different elements as concrete (index c), surface reinforcement (index Bst) and anchor plate (index AP). The stiffness of the springs is specified with \(\bar{E}_i = \frac{E_i A_i}{l_i}\).
8.5 Validation of the analytical model

The analytical spring model can be assembled to a truss model. Each beam describes a row of headed studs.

\[
\begin{align*}
C_{AP,ges} & = \left( \frac{1}{C_{AP}} + \frac{1}{C_{Du} + C_{Haft}} \right)^{-1} \\
C_{AP} & = \frac{E_s \cdot A_{AP}}{L} \\
C_{Du} & = \frac{E_s \cdot A_b}{L} \\
C_{Haft} & = \begin{cases} 
\infty & \text{with adhesive bond} \\
0 & \text{without adhesive bond} 
\end{cases}
\end{align*}
\]

(15)

The analytical model can be calculated with the state of equilibrium. The acting Load N deforms the truss model unsymmetrical because of the anchor plate located at one side. The calculation has shown that an acting static load of 0.6 MN has no significant results for a duration of 8 hours. For a consideration of 50 years shrinkage is a main influence on anchor plates but the results of the investigation show that it leads a total additional stress of about 3.0 kN per fastener. Furthermore, the short time relaxation is on the safe side for this analytical model. Creeping and shrinkage has a small effect on large anchor plates with additional consideration of an adhesive bond between anchor plate and concrete.

References:


STEEL-TO-CONCRETE JOINTS WITH LARGE ANCHOR PLATES UNDER SHEAR LOADING

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ABSTRACT

In industry and plant engineering especially, high flexibility is required at joints between steel and concrete. According to current standards, the maximum number of fasteners is limited to an arrangement of 3 x 3 anchors on an anchor plate. The load-carrying behaviour of large anchor plates under tension, shear and restraining forces was investigated within the scope of the research project “Large Anchor Plates with Headed Studs for Highly Stressed Constructions in Industry and Plant Engineering”. This paper describes the research results obtained at the University of Stuttgart for large anchor plates under shear loading. Findings for large anchor plates under tension and restraining forces were mainly investigated at the University of Kaiserslautern20.

The shear behaviour and distribution of forces within the anchor plate are crucial issues in the development of a design concept for steel-to-concrete joints with large anchor plates. The influence of different parameters, such as the dimensions of the anchor plate, the embedment depth of the headed studs or the eccentricity of the shear force, have been studied in several test series. Supplementary reinforcement was placed close to the headed studs in tension to strengthen the load-carrying capacity of the joint. The distribution of the shear forces has been assessed by means of numerical investigations together with the influence of further parameters such as concrete strength and reinforcement ratio. With regard to the load distribution within the anchor plate, elastic and plastic design approaches have been taken into account.

Based on these studies, an efficient design model has been developed for steel-to-concrete joints with large anchor plates and a higher number of fasteners than originally permitted. Owing to the consideration of supplementary reinforcement, in addition to the plastic design approach, an alternative economical solution is given which links the component method according to EN 1993-1-84 to the fastener technique according to EN 1992-45.
1 Introduction

Recently, hybrid building technology, where both materials, steel and concrete, are used according to their properties and their most suitable application, has spread among buildings and industrial practice, such as steel frames connected to column bases on foundations, or steel or composite beams joined to the concrete walls of staircases. Recently, several design concepts have been developed for steel-to-concrete joints within national and international research projects. Design approaches based on the component method have gained acceptance, where concrete and steel components are considered simultaneously.

In industry and plant engineering, large forces have to be transferred from structural steel elements to concrete elements such as walls or foundations. As a consequence of this, large anchor plates are required which enable the activation of a sufficiently large concrete volume to support these loads. However, current design rules limit load-carrying capacities due to the limitation of nine fasteners per anchor plate and the elastic design concept, which is the dominant method for the design of fastenings. Therefore, the resistance of an anchor plate is limited, which leads to complicated load distribution arrangements over several single anchor plates or solutions in pure concrete that avoid the “difficult” interface between steel and concrete. The scope of the research project was the development of a design model for large anchor plates with more than nine headed studs per anchor plate. Tests under normal, shear and restraining forces were conducted within the scope of the project (Fig. 1).

2 State of the art and design concepts

The design concept for large anchor plates was developed based on the component method which distinguishes between different failure modes based on a characterized structural behaviour of the single components. In an overall design concept, steel components according to EN 1993-1-8 and concrete components according to EN 1992-4 are considered simultaneously. In addition to the load-carrying capacity of the joint, the load-deformation behaviour can be determined for the whole joint.

In the design of the joint, it is possible to strengthen certain, more flexible components, thus achieving ductile behaviour of the joint. For example, if the thickness of the anchor plate is varied, the flexible behaviour of the anchor plate, which is considered within the T-stub under tension and compression according to EN 1993-1-8, can come into play. On the other hand, higher resistances for some concrete failure modes can be reached if, for example, supplementary reinforcement is placed next to the headed studs in tension and the interaction between concrete and reinforcement is considered according to recent approaches.

Figure 2: Elastic (left) and plastic (right) distribution of forces
The more detailed analyses of the load-carrying behaviour of the single components, their individual
stiffnesses and their interaction enable assumptions to be made regarding the loading side for the
distribution of loading which are significant for the design of anchor plates. Internal forces such as
normal, shear and bending forces are distributed among the rows of headed studs according to an
elastic design concept if the anchor plate can be assumed to be sufficiently rigid (Fig. 2). As part of
the elastic design concept, it is assumed that all fasteners have the same stiffness. Load redistribu-
tions in the area of the anchor plate should not be considered, as the fastener carrying the maximum
load is critical for the load-carrying capacity of the anchor plate. The elastic design concept is mainly
used for the design of fasteners because it is especially suitable for concrete failure modes. Due to
the reduced deformation capacity of the pure concrete components, load redistributions are neglec-
ted. Plastic redistributions among the rows of headed studs may be taken into account if brittle con-
crete failure modes are excluded. In the case of flexible anchor plates, a non-linear distribution of
forces is possible, based on the requirement of a technical report referenced by EN 1992-45.

A plastic design concept was derived for anchor plates with several rows of fasteners within the
scope of the first investigations of the ductile behaviour of steel-to-concrete joints. Besides the non-
linear distribution of normal forces, this concept allows shear forces to be distributed according to
the utilization factor based on the interaction equations of each row. This is contrary to the elastic
approach, where the shear forces are distributed equally among the rows. Limiting cases for the dis-
tribution of shear forces may be derived from equilibrium conditions for different eccentricities.
With a large eccentricity of the load, for example, the whole shear force can be carried by friction.
For smaller eccentricities, shear loads are carried by friction and by a contribution of the rows of
fasteners on the loaded side, and in the case of very small eccentricities, the shear force is distributed
equally among the rows of fasteners. An improved approach was developed for plastic design in Lot-
ze et al. For a full utilization based on the interaction of normal and shear forces, sufficient de-
formability should be available at the anchor plate and its fasteners.

3 Experimental investigations

3.1 General

Firstly, experimental investigations were carried out to gather information on the behaviour of large
anchor plates under shear loading and to apply these results as validation for further numerical inves-
tigations. In a second step, an analytical design model that takes into account the load distribution in
the steel-to-concrete joint was developed based on the experimental and numerical studies. Four-
teen tests were carried out on large anchor plates under shear loading without edge effects. The fol-
lowing parameters were varied within the specified limits:

- Eccentricity \( e \) (80 and 1000 mm)
- Thickness of anchor plate \( t_{ap} \) (15 and 40 mm)
- Length of headed stud \( h_n \) (100 and 250 mm)
- Cracked and uncracked concrete
- Reinforcement ratio of supplementary reinforcement (1 \( \phi 8 \) mm and 2 \( \phi 8 \) mm)

By varying the parameters given in Table 1, concrete and steel failure modes were planned beforehand
and achieved in testing. In order to identify the influence of the different parameters, a testing matrix
was chosen in a way that reference tests had been defined from which the other tests differed by only one single parameter. Thus, the influence of the various single parameters could be clearly identified.

Table 1: Test parameters of shear tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Headed stud</th>
<th>Eccentricity e [mm]</th>
<th>Thickness of anchor plate [mm]</th>
<th>Supplementary reinforcement per headed stud</th>
<th>Concrete state</th>
</tr>
</thead>
<tbody>
<tr>
<td>B3-Q</td>
<td>SD16/100</td>
<td>80</td>
<td>40</td>
<td>1 ø8 mm</td>
<td>cracked</td>
</tr>
<tr>
<td>R2-1Q</td>
<td>SD16/100</td>
<td>1000</td>
<td>40</td>
<td>2 ø8 mm</td>
<td>uncracked</td>
</tr>
<tr>
<td>R2-2Q</td>
<td>SD16/100</td>
<td>80</td>
<td>40</td>
<td>2 ø8 mm</td>
<td>uncracked</td>
</tr>
<tr>
<td>R2-3Q(1)</td>
<td>SD16/100</td>
<td>1000</td>
<td>40</td>
<td>2 ø8 mm</td>
<td>cracked</td>
</tr>
<tr>
<td>R2-3Q(2)</td>
<td>SD16/100</td>
<td>1000</td>
<td>40</td>
<td>2 ø8 mm</td>
<td>cracked</td>
</tr>
<tr>
<td>R2-4Q(1)</td>
<td>SD16/100</td>
<td>80</td>
<td>40</td>
<td>2 ø8 mm</td>
<td>cracked</td>
</tr>
<tr>
<td>R2-4Q(2)</td>
<td>SD16/100</td>
<td>80</td>
<td>40</td>
<td>2 ø8 mm</td>
<td>cracked</td>
</tr>
<tr>
<td>R3-1Q</td>
<td>SD16/250</td>
<td>1000</td>
<td>40</td>
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<td>40</td>
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<td>2 ø8 mm</td>
<td>uncracked</td>
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</tbody>
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The tension loading on the headed studs on the unloaded side of the anchor plate was influenced by the variation of the eccentricity. In tests with a large eccentricity, higher tension forces were obtained due to the resulting higher bending moment at the anchor plate. For this reason, different values of eccentricity were used. In addition, the thickness of the anchor plate was varied in order to determine the influence of a flexible anchor plate.

Where a concrete cone developed due to tension loading on the headed studs, these forces could be tied back by supplementary reinforcement, and, as a consequence, the load-carrying capacity of the related concrete failure modes increased. The supplementary reinforcement was placed next to the headed studs as shown in Fig. 3 based on previous research results\(^\text{10}\). In test B3-Q, one leg of a stirrup was placed next to the headed stud, whereas one stirrup with two legs per headed stud was considered in all other tests. In addition, several tests were performed in cracked concrete. Metal sheets were placed near the headed studs in tension and a crack width of approx. 0.3 mm was achieved on the axis of the two rows of headed studs on the unloaded side (Fig. 3).
3.2 Loadbearing behaviour and failure modes

The tests under shear load were conducted at the Materials Testing Institute, University of Stuttgart (Materialprüfanstalt Universität Stuttgart). To verify the analytical model, different failure mechanisms, such as concrete or steel failure, were initiated by varying the parameters given above (Fig. 4).

Concrete pry-out failure was observed in the series with short headed studs, for tests with small and large eccentricity (Figs. 4a and 4c). An elastic distribution of the normal forces was present in these tests, as the deformation of the anchor plate itself was very small. The anchor plate cross-section remained plane and no plastic deformations occurred. The embedment depth of the supplementary reinforcement in the concrete cone was small in comparison to the tests with long headed studs. Nonetheless, the supplementary reinforcement was activated. In the test with a smaller reinforcement ratio (B3-Q), the load-carrying capacity was 20 % lower (Fig. 5b) than in the test with high reinforcement ratio (R2-2Q). In the test series with cracked concrete, the load-carrying capacities were reduced by 20 % compared with the reference tests (Figs. 5a and 5b).

The tests with longer headed studs and a thicker anchor plate (Fig. 4b) resulted in a pry-out failure with a large concrete cone. The supplementary reinforcement was activated and the yield stress of the reinforcement was reached. In the tests with a small eccentricity, high shear forces occurred and the joint failed due to the headed studs shearing off. The influence of the pre-induced cracks was negligible, as many radial cracks developed in cracked and uncracked concrete. Only small deformations of the thick anchor plate were measured in these tests. In the rows of headed studs in tension, plastic redistributions occurred, measured by the strain gauges, due to the large uplift on the unloaded side of the anchor plate.
In the tests with thin anchor plates, the influence of the thickness of the anchor plate was observed by means of the development of yielding mechanisms (Fig. 4d). In the tests with a large eccentricity, yielding mechanisms developed in the tension zone of the anchor plate. At maximum load, a mixed failure occurred due to yielding in the anchor plate and yielding in the headed studs (Fig. 6). In the tests with small eccentricity and thin anchor plate, the headed studs sheared off at the base of the shaft similar to the tests with a thick anchor plate.

### 3.3 Load distribution of normal forces

One of the main goals of the experimental investigations was to determine the distribution of normal forces in the different rows of headed studs by varying the eccentricity of the shear force. To do that, strain gauges were attached to the shafts of the headed studs, to the supplementary reinforcement and to the top of the anchor plate. The stresses were evaluated by means of strains. In the rows of headed studs, the strain gauges were located according to a chessboard pattern. They were attached to the middle of the shaft in order to avoid the influence of the bending. The forces in the rows of headed studs were determined as mean values of the results measured by the strain gauges based on the mechanical properties of the material. An elastic distribution of normal forces was monitored in the tests with short headed studs and concrete failure modes. On the other hand, in the tests with long headed studs, plastic redistribution was measured towards the intermediate rows of studs. Compression forces were determined up to the maximum load in the first row in all tests with large eccentricity (Fig. 7). The values measured by the strain gauges for large anchor plates and short headed studs comply with the definition of load distribution within anchor plates according to Eliegehausen et al.\(^7\), which assumes an elastic distribution of forces for brittle failure modes such as concrete pry-out failure. An elastic distribution of forces was determined in the tests with short headed studs (Fig. 7a). Plastic redistribution among the rows of headed studs was observed for ductile failure modes such as steel failure in test R5-1Q (Fig. 7b).

### 4 Numerical investigations

The aim of the numerical investigations of the shear tests was to recalculate the maximum loads and the failure modes of the tests. Based on the tests, the numerical model was validated and the load distributions in the anchor plate evaluated – specifically, the

![Figure 7: Load distribution of normal forces: a) test R2-1Q and b) test R5-1Q](image_url)

![Figure 8: Comparison of maximum loads of FE model and test results for a) large eccentricity and b) small eccentricity](image_url)
distribution of shear forces among the rows of headed studs. In addition, parameter studies were conducted. The investigations of the anchor plate considered various parameters, including thickness, headed stud embedment depth and shear force eccentricity. The numerical results allowed also the enlargement of the database for the validation of the analytical model. The non-linear FE program MASA (Macroscopic Space Analysis) was used for the numerical studies as it permits modelling of the localized damage behaviour of the concrete in the shear joints.

Good correlation was observed between the maximum loads from the tests and the numerical results. Figs. 8a and 8c compare large and small eccentricities. The numerical model was also able to reproduce the load-carrying behaviour of different failure modes such as steel and concrete failure for the same assumptions concerning boundary and modelling conditions. The principal tensile strains are given in Fig. 9, for which the crack propagation was recalculated by the FE model. These results are in accordance with the failure mechanisms shown in Figs. 4a and 4b. The non-linear behaviour of the steel was considered by the von Mises yield criterion and the damage behaviour of the concrete by the microplane model implemented in MASA.

A number of parametrical studies were performed in order to examine the load-carrying behaviour of the anchor plates and to quantify the influence of different factors such as geometry and material properties. The parameters were selected so that the upper and lower values were validated by test results and the database was enlarged by parameter studies for values in between those boundaries. In addition, parameter studies were also conducted beyond the configurations of the test specimens. The concrete strength and the reinforcement ratio were varied in addition to varying the parameters eccentricity and anchor plate thickness. The studies of the influence of eccentricity and anchor plate thickness are shown in the following. A detailed description of all parameter studies is given in the final research report.

The distribution of shear forces among the different rows of headed studs was quantified by the numerical investigations, because in the tests, it was not possible to determine the shear force per stud in a similar way as for the normal forces. Therefore, the nodal forces, derived from the numerical calculations, were summarized in a section through the base of the headed studs in the different rows and compared with each other. The distribution of shear forces is illustrated in Fig. 10 by the principal compressive stresses in the concrete. The results of the different principal compressive stress plots are scaled equally to the range between 0 and -10 N/mm² in order to be able to compare the plots with each other.

Varying the eccentricity of the shear force has an influence on the load distribution of the tensile forces on the unloaded side and in the distribution of the shear forces, since large deformations occur due to the tension uplift of the anchor plate in the area of the T-stub (P1-2 and P1-14). High shear forces were determined in the shafts of the headed studs, especially with a thin anchor plate. For the studies with
large eccentricities, a high contribution by the friction forces was determined for the shear capacity due to the comparatively high compressive forces at the anchor plate. In these cases the shear force is mainly transferred by friction and the first row of studs. The distribution of the shear forces, determined in numerical studies with a small eccentricity, is approximately similar for thick and thin anchor plates. The shear force is distributed equally among the rows of headed studs (P1-5 and P1-17). The observations of the load distribution of the shear force, based on the numerical studies, are generally in line with the current plastic design concept for steel-to-concrete joints. However, high tensile and shear forces may occur due to membrane effects in the tension zone of the anchor plate. These effects require a common design concept for steel and concrete components.

5 Component model for steel-to-concrete joints in shear

5.1 General
In the design concept of the component method, the design resistances of the single components are compared with their actual loading accordingly. In addition, interaction equations due to loading in different directions have to be considered. Within the scope of a design concept for steel-to-concrete joints under shear loading, simplifications have to be considered, especially for anchor plates with a higher number of headed studs due to the high degree of static indeterminacy. A simplified analytical design model is proposed for steel-to-concrete joints with large anchor plates. In this model the tensile
forces in the rows of headed studs on the unloaded side are assembled into one resultant tensile force. The analytical design model for the tension component has been calibrated and verified by test results under normal forces at the University of Kaiserslautern. On the loading side, an elastic design concept with brittle failure mechanisms may be considered as well as a plastic design concept with load redistributions corresponding to the large deformations of the steel components.

5.2 Single components for tension and shear

The component model for the steel-to-concrete joints is based on existing design concepts for the failure modes of the single components. The concrete capacity design for the current concrete components is expanded by more current approaches that consider the reinforcement near the fasteners. The analysis of the concrete components have to consider, on one hand, the fastener with the highest load and, on the other, the group failure mechanisms. Simplified design approaches were developed, where the resistances of the supplementary reinforcement and the concrete are combined. For the steel components within the anchor plate, the T-stub model for tension and compression loading is considered, which has been extended by newer concepts for T-stubs with several rows of bolts.

5.3 Component model for large anchor plates under shear

In the following, an analytical design model is proposed for steel-to-concrete joints under shear loading which considers the different requirements and assumptions concerning the load distribution and the resistances depending on the different failure mechanisms. The eccentric loading of a steel-to-concrete joint is split into a bending moment for the definition of the normal components and into a pure shear force for the determination of the shear components, see Fig. 11. This subdivision is applied to all failure modes. Of course, both loading directions are considered for the interaction equations.
Furthermore, the steel and concrete failure mechanisms are differentiated. Therefore, it is possible to distinguish between a single failure of the most heavily loaded fastener for steel failure and a group failure for concrete failure. In contrast to EN 1992-4\textsuperscript{5}, the components of the reinforcement are associated with the group failure mode because the recent approach for the load-carrying capacity of the reinforcement\textsuperscript{1,10} considers the interaction between steel and concrete. So the resistance of the concrete cone is considered for the strength of the characteristic concrete component which should only be determined by considering the group of fasteners. For the tensile components, increases of the resistance are considered due to the overlap of the compression and tension zones\textsuperscript{23}. If elastic distribution of normal forces is assumed, effects due to an eccentric loading are included in the calculation of the tensile resistance for concrete failure\textsuperscript{5}. Within the scope of the plastic design concept, if brittle failure mechanisms can be excluded, the loading may be distributed equally among the rows of headed studs assuming plastic deformations. The tensile loading may be assigned to the rows of headed studs on the unloaded side for steel failure mechanisms, and the shear force that cannot be carried by the friction components is assigned to the first row of studs (Fig. 11).

The normal forces in the rows of headed studs on the unloaded side are determined based on the equilibrium of vertical forces and moments around the compression zone (Fig. 2). The internal forces can be distributed within the anchor plate according to Fig. 11 for the different failure modes. The plastic design concept can be applied, if steel failure is the governing failure mode and the interaction equations for the plastic design given in Figure 11 are considered. Initially, an assumption is made for the depth and location of the compression zone and the number of rows of headed studs in tension. Within an iterative calculation, the assumption is reviewed and probably updated. A step-by-step procedure for designing the steel-to-concrete joints under shear loading and a general procedure are given in the project report\textsuperscript{14}. In addition to a simplified design model, where the tension loading is only assigned to the two rows of headed studs on the unloaded side, a more detailed model implemented in design software is described\textsuperscript{14}. Consequently, load redistributions and the flexible anchor plate can be considered based on the displacement method using the stiffness of the headed studs\textsuperscript{10}.

### 5.4 Verification of the component model

For the tests with large eccentricity, the results from the analytical design model for the different failure modes agree sufficiently well with the maximum test loads. The actual failure modes are also reproduced by the design concept (Fig. 12a). The comparison between the results of the design model and the tests with small eccentricity show a conservative approach to the load-carrying capacities (Fig. 12b). For the tests with small eccentricity and short headed studs, the resistance of the concrete pry-out failure is the governing failure mode in the analytical design model. As the supplementary reinforcement in the area of the headed studs cannot yet be considered within the model.

![Figure 12: Comparison of maximum loads of the analytical model and the experimental results for a) large eccentricity and b) small eccentricity](image-url)
for pure shear, increases in the load-carrying capacity of this component will be possible in the future development of an improved model. The database for validating the analytical model was enlarged with the help of the numerical parameter studies. Further influences of different parameters were considered. In addition to the validation of the analytical model in comparison to the 14 test results, approx. 90 additional numerical investigations were used for verification

6 Conclusions

Results of a research project were presented, where design models for large anchor plates under shear, tension and restraining forces had been generated. These models were developed based on experimental and numerical investigations for normal and restraining forces and shear forces. One aim of the project was to determine the load distribution of forces at the different rows of studs on the anchor plate, measured in the tests by strain gauges on the shafts of the headed studs and obtained by numerical calculations. Besides the embedment depth of the headed studs, other parameters were varied such as eccentricity, thickness of anchor plate and the reinforcement ratio of the supplementary reinforcement. In addition, tests were performed in cracked and uncracked concrete. Extensive numerical parameter studies were performed in order to enlarge the database of the tests and to determine the distribution of shear forces among the rows of headed studs. The numerical models were validated by the test results. The distribution of shear forces among the rows of headed studs could be quantified based on the numerical studies. The load-carrying behaviour and the influence of different parameters were examined with the help of the experimental and numerical studies. It might be possible to expand existing design concepts for large anchor plates if the requirements of an elastic or plastic design are considered consequently. Therefore, the resistances of the different failure modes of the single steel and concrete components were used and extended by recent approaches to take into account the supplementary reinforcement. Further studies of the load-carrying behaviour of large anchor plates were performed. Edge effects are not yet considered in the design model and have to be conservatively estimated by applying all loads to the row of headed studs close to the edge. Further numerical and experimental investigations have to be carried out for large anchor plates with edge effects in order to estimate the load-carrying behaviour and the load redistributions.

7 Acknowledgements

This IGF project of the Deutscher Ausschuß für Stahlbau (DAST) was funded by the AiF within the scope of the cooperative Industrial Research Programme (IGF) by the Federal Ministry for Economic Affairs and Energy based on a decision by the German Bundestag. The authors would like to take this opportunity to express their gratitude to all supporters of their research. Special thanks go to the industry partners Köster & Co. GmbH and Rau-Betonfertigteile GmbH & Co. KG for supplying the test specimens. Moreover, the authors would like to thank the project partners from the University of Kaiserslautern for the good cooperation and the Materials Testing Institute, University of Stuttgart (MPA), for their support regarding the tests.
References:


BONDED OVERLAY STRENGTHENING OF HOLLOW CORE SLAB
WITH AND WITHOUT INTERFACE SHEARKEYS CONNECTION

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ABSTRACT

Precast Prestressed Hollow Core Slabs (PPHCS) are most commonly used as flooring and roofing elements. Usually, a new layer of concrete is placed on the top of hollow core slabs to create a continuous and levelled surface. The common thickness of this bonded overlay will be around 50 mm to 75 mm deep. The provision of Bonded Overlay (BO) will increase the cracking load and flexural strength of hollow core slab after the full composite action is developed. In the present study, the effect of shear keys at the interface of bonded overlay and hollow core slab is studied. The hollow core slab and bonded overlay is expected to have a full composite action until failure without any interface separation. The dimension of hollow core used in this investigation is 600mm wide, 150mm depth and 3500mm length. In total, three full-scale hollow core slabs were tested under shear span (a) to depth (d) ratio of 7.5. The three specimens which include un-strengthened slab denoted as control slab, slab strengthened with bonded overlay without any shear keys at the interface and bonded overlay with shear keys. Bonded overlay specimens without shear keys resulted in interfacial failure and it was able to increase the peak load by 38.4% compared to the control specimen. However, the bonded overlay with shear keys resulted in full composite action till the final failure and it was able to increase the peak load by 59.6% compared to the control specimen. The provision of shear keys at the interface of hollow core slabs and bonded overlay resulted in full composite action.

1 Introduction

Precast Prestressed Hollow-Core Slabs (PPHCS) are the precast members which are commonly used as roof and flooring units in residential and commercial buildings, parking structures, short span bridges. PPHCS consist of continuous interior recesses running along its length. These recesses significantly reduce the self-weight of the slab. Good improvement in flexural and shear strength of HCS can be achieved using bonded overlay technique. A thin layer of concrete (bonded overlay) of appropriate thickness can be placed in the compression region to attain desired structural behavior. However, the horizontal shear transfer between concrete interfaces should be sufficient to allow full composite action under flexure loading. Both ACI¹ and PCI² code provisions suggest that a minimum roughening amplitude of 6.3 mm is required for attaining full composite action. Previous studies by authors have focused on the FRP strengthening of prestressed hollow core slabs³-⁷. However, a very few investigations in the past have focused on interfacial shear behavior of composite hollow core slabs. Mones and Brena⁸ performed push off test on hollow core composite
section and the authors concluded that roughened interface develops higher horizontal shear strength and less horizontal slip compared to machine finished specimen. They also noted that the roughening is more effective when the grooves are perpendicular to applied loads. Girhammar and Pajari\textsuperscript{9} performed pull off test to determine the interfacial shear strength of different topping materials such as normal and fiber reinforced concrete. The authors concluded that interfacial strength was slightly higher for slabs with fiber reinforced concrete topping. Studies focusing on the flexural behavior of composite hollow core slabs concluded that full composite action is achieved up to ultimate load.

2 Research Significance

The placing of bonded overlay on the top PPHCS is a most common technique in the precast buildings. The positive impact of bonded overlay on the strength and serviceability of hollow core slabs will come into the picture once it develops the full composite action with parent concrete. Hence, it is important to understand the complete flexural behaviour of prestressed hollow core slabs strengthened with bonded overlay using with and without shear keys at the interface.

3 Experimental Work

3.1 General

Three full scale PPHCS were tested under flexure to understand the composite action of the bonded overlay strengthened specimen. To understand the composite action between bonded overlay and hollow core slabs, slab was strengthened with only bonded overlay strengthening and another specimen was strengthened with bonded overlay with interfacial shear keys. All three specimens were tested under shear span depth ratio of 7.5. Figure 1 explain the test setup. Details of the test specimens are given in Table 1.

3.2 Material Properties

3.2.1 Concrete

All the prestressed precast hollow core slabs were manufactured in the precast plant. The average 28 day cube compressive strength of these slabs was 43 MPa. The unit weight of concrete used was 2400 kg/m\(^3\). The concrete used for bonded overlay had an average 28 day compressive strength of 37 MPa. Care was taken to have a more or less similar concrete strength for both the parent slab and the bonded overlay to ensure monolithic behavior.

3.2.2 Prestressing strands

Three numbers of seven wire (9.53 mm nominal diameter) low-relaxation strands were used. Coupon tests were conducted and the average ultimate tensile strength and modulus of elasticity was found to be 1860 MPa and 196 GPa respectively. The strands were anchored at one end and stressed from the other end to an effective stress of 980 MPa.
Table 1 - Details of Test Specimens

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<th>BO Concrete Strength (MPa)</th>
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3.3 Strengthening Procedure

Usually, a new concrete overlay (screed) is placed on top of hollow core slab at the site to create a continuous levelled surface. The thickness of this overlay will be typically about 50 mm. Therefore, to resemble the actual site conditions, 50 mm overlay thickness was placed on the top surface of hollow core slabs. Ensuring proper bond between the overlay and parent concrete surface is very difficult without any shear keys. To ensure proper shear transfer, minimum number of shear keys were provided as per ACI 318 to prevent bond failures and to ensure complete composite action till the ultimate strength. The L-shaped shear keys were placed at 300 mm center to center along the length of hollow core slab. A hole of 8-mm diameter was drilled up to 25 mm depth to place the shear keys. After installation of shear keys, the holes were filled with epoxy resin to ensure bonding of shear keys to the parent slab. Fig. 2 shows the provision of shear keys at the interface of hollow core slab and the bonded overlay. Bonded overlay slabs were tested after proper curing of the overlay for a minimum of 28 days.

3.4 Test Setup and Instrumentation

Two Linear Variable Displacement Transducers (LVDT) of 100 mm stroke were placed to measure the deflection at the mid-span. Two additional LVDT’s of 50 mm stroke were placed at one-third of the span to measure the deflection as shown in Figure 3 (a). Two strain gauges (Pl-60) of 60 mm gauge length were placed at the top and bottom of the concrete slab to measure the concrete surface
strain variation during the testing. Four strain gauges (UFLA-5) of 5 mm gauge length were installed on prestressing strands at mid-span and at one-third of the span as shown in Figure 1(b).

1) MTS 250 kN  2) spreader beam 3) Point Loadings 4) Hollow core slab 5) DAQ 6) DIC light source 7) DIC camera 8) DIC system 9) MTS System

Figure 2: Test setup of Hollow Core Slabs

(a) Placing of LVDT’s
(b) Strain gauge on strands

Figure 3: Instrumentation Details
4 Experimental Results

The experimental results observed were presented in Table 2. The overall behavior of individual specimens are described in the sections below.

4.1 Control Specimen (7.5-C)

The load-displacement response of the control specimen is illustrated in Figure 4. The first flexural crack was observed at mid-span in the constant moment zone when the load reached to 25 kN. Cracking was also reflected through the change in the slope of the load-displacement curve. With the further increase in the applied load, strands significantly contributed to the load resisting mechanism. Specimen attained a peak load of 50.7 kN, and a corresponding displacement of 44.6 mm. The load resistance was slowly declining with the further increase in load level and the slab failed by crushing of concrete in compression at a displacement of 93 mm.

4.2 HCS with Bonded Overlay (7.5-HCS+BO)

HCS+BO was strengthened specimen, which is having concrete bonded overlay of 50mm in the compression region. This slab has also tested at same shear span to depth ratio of 7.5 to find the effectiveness of the overlay. From concrete bonded overlay, it was observed that the ultimate load of the strengthened specimen was 59.2 kN with the deflection of 13.4 mm and ultimate deflection measured was 85 mm. Failure progression was as follows; First crack observed at the bottom over the constant moment zone; sudden horizontal separation between bonded overlay and hollow core slab; the yielding of strands; finally crushing of concrete at compression region.

4.3 HCS with Bonded Overlay and Shear Keys (7.5-HCS+BO+SK)

HCS+BO+SK was strengthened specimen which is having a thin layer concrete bonded overlay with shear keys at the interface to prevent horizontal shear failure. This slab has also tested at same shear span to depth ratio of 7.5 to find the effectiveness under flexure. From concrete bonded overlay, an increase in the cracking moment and post cracking stiffness was observed whereas from the shear keys it was observed that it was very effective in preventing the horizontal shear failure. The ultimate

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<th>Change in PL (%)</th>
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Table 2: Test Results of HCS
load of strengthened specimen was 80.9 kN with the deflection of 56.7 mm. Failure progression was similar to that of control specimen (flexure-shear mode).

Figure 4 shows the comparison of load-displacement of all tested slabs. The hollow core slab strengthened with bonded overlay without any shear keys was able to increase the load by 38.3% after an interfacial shear failure. A sudden drop in the load was observed after which the strengthened slab behaved like a control specimen. The slab strengthened with bonded overlay and shear keys at the interface was able to increase the load by 59.6% without compromising the post-cracking stiffness behavior. The table 3 summarizes the calculations for cracking moment, failure load for the control and strengthened slabs. Moment capacities are calculated by developing moment-curvature for the cross sections. Shear capacity (Ps) predictions are calculated based on ACI 318-05 specifications. Finally, the predicted response are compared with experimental test results and a good correlation was observed between them.

4.4 Comparison

Figure 4 shows the comparison of load-displacement of all tested slabs. The hollow core slab strengthened with bonded overlay without any shear keys was able to increase the load by 38.3% after an interfacial shear failure. A sudden drop in the load was observed after which the strengthened slab behaved like a control specimen. The slab strengthened with bonded overlay and shear keys at the interface was able to increase the load by 59.6% without compromising the post-cracking stiffness behavior. The table 3 summarizes the calculations for cracking moment, failure load for the control and strengthened slabs. Moment capacities are calculated by developing moment-curvature for the cross sections. Shear capacity (Ps) predictions are calculated based on ACI 318-05 specifications. Finally, the predicted response are compared with experimental test results and a good correlation was observed between them.
Table 3 - Comparison between the predicted and the experimental results

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</tbody>
</table>

(*Note: $P_{cr}$=Load at first crack; $P_{fc}$= Flexure load capacity of the specimen; $P_s$=Shear load capacity of the specimen; $P_u$=Ultimate load of the specimen governing failure.)

5 Conclusions

Three full-scale prestressed hollow core slabs were tested to investigate the composite action between the hollow core slabs and the bonded overlay. The main conclusion drawn from this study are as follows:

- The hollow core slab strengthened with only bonded overlay without any shear keys at interface increase the strength by 38.3%. The specimen had a failure at the interface due to propagation of shear crack through them.
- The bonded overlay strengthened specimen with interfacial shear keys increased the peak load by 59.6% and full composite action was achieved. The interfacial shear failure of HCS were prevented and the specimens finally failed in a flexure-shear mode.
- Shear keys are effective to resist the interfacial horizontal shear force. As a result, they helped the specimen in preventing the propagation of shear crack at the interface which was considered as the weak link of the composite section.
- The analytical predictions calculated as per ACI code to predict the capacity of the hollow core slab with and without overlay had a good correlation when compared to the experimental results.

6 Acknowledgement:

This experimental research was sponsored through Prime Minister Fellowship schemed initiated by Confederation of Indian Industries (CII) and Department of Science and Technology, India. The authors gratefully acknowledge their generous support. Specimens were cast at casting yard of PRECA Ltd. Their generous support of materials towards this research is also thankfully acknowledged.
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CONNECTIONS UNDER SPECIAL LOADINGS:
FIRE, ELEVATED TEMPERATURE
BACKGROUND ON THE FIRE EVALUATION OF POST-INSTALLED REINFORCEMENT BARS IN CONCRETE

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ABSTRACT

The progress in the field of polymer adhesives has encouraged their application for bonding steel bars into concrete for structural purposes. However, high temperatures have the effect of weakening the bond and endangering the construction under a fire situation. Consequently there is a need for the evaluation of the performances under fire situation. This paper summarizes the different investigations that took place in order to establish a European fire assessment method. The justifications on the test procedure to characterize bond resistance at high temperature are presented with the supporting research. The paper describes some thermo-mechanical phenomena occurring at high temperature (such as water vaporization or polymer post curing). The methodology for mortar qualification and design in a fire situation (using thermal fields) is explained. In parallel, the decisions resulting from the harmonization with pre-existing evaluation and design guides at ambient temperature are described.

1 Introduction

The post-installation of reinforcement bars in already cured concrete members is performed by bonding a rebar with a mortar inside a drilled hole. The structural adhesives used are generally bi-components polymeric mortars (most of the time based on epoxy or vinyl ester chemistries) and cure directly at ambient temperature on the construction site. Mechanical bonding performances are generally reached in less than 24 hours after the injection of the adhesive allowing fast installation of rebars for the repair or extension of already existing structures.

Post-installed rebars (PIRs) are used either as anchors (to connect a slab to a wall for example) or as overlap joints (to connect two slabs together for example) in which case the goal is to transfer tensile stress to a neighboring overlapping cast-in rebar. For both applications, the PIR system (concrete, mortar, rebar) is assessed in tension only. The evaluation method at ambient temperature was described in the Technical Report n°23 (TR023)1. Since 2004, research2 in the fields of adhesive behavior exposed to heat has led to develop an evaluation method at high temperature in a fire situation. This fire evaluation method was established within an EOTA Expert Group between 2012 and 2014. With the implementation of the new European Construction Product Regulation (CPR)3 in
July 2013, both the ambient temperature and fire assessment methods were harmonized in a unique European Assessment Document (EAD)\(^4\).

This article provides the background information used to establish the fire evaluation method described today in the EAD\(^4\). The goal is to communicate the origin of the guide by presenting the research conclusions and arguments discussed within the EOTA Expert Group. The method was established in accordance with existing guides such as the TR023\(^1\) (for assessment of PIR at ambient temperature) and Eurocode 2, part 1-1, section 8\(^5\) (for the design of cast-in rebars). The paper presents the background of i) the test method at high temperature, ii) the fire evaluation method and iii) a design method proposal. The two first parts (i and ii) are currently exposed in the EAD\(^4\).

## 2 Test Method

### 2.1 Samples

The test method for fire evaluation is described in part 2.2.3 of the EAD\(^4\). The goal of these tests is to determine the variation of the experimental bond resistance \((f_b)\) versus temperature \((\theta)\) for a given adhesive. This is achieved by pullout tests at high temperatures. The tests are carried out on concrete cylinders (diameter 16 cm and height 250 cm) encased in a steel pipe (5 mm thick) in order to prevent splitting failure during the pullout (Figure 1). The surface of the concrete is confined with a steel plate to prevent concrete cone failure (in accordance with EOTA Technical Report TR048\(^6\)).

The rebar of diameter 12 mm is embedded on a length \(l_v=120\) mm in the axial direction.

![Figure 1: High temperature pullout tests sample geometry](image)

Two thermocouples (TC1 and TC2) are positioned respectively at 1 cm from the top and at the bottom of the bond. This is done because thermal gradients along the bond length may corrupt the pertinence of failure temperature used in the evaluation. It can be noted that, unlike an anchor, a PIR cannot be directly exposed to fire since it is protected by a concrete cover on all sides. The rebar should not be the heat vector for increasing the bond temperature. To ensure that the main heat flux is applied on the lateral sides of the sample (which cannot be quantified systematically through simple measurement), a 10°C maximal temperature difference between TC1 and TC2 is imposed to ensure thermal uniformity along the bond. This criterion is only applied up to 50°C after which water vaporization disturbs thermal fields (as described in part 4.2).
2.2 Test Procedure

Two test procedures have been studied to establish the variation of bond resistance with temperature ($f_b-\theta$). In the first procedure (referred to as the ‘constant load’ procedure), a load is applied on the rebar and remains constant during the progressive heating of the sample. Failure occurs when a given temperature is reached along the bond. In the second procedure (referred to as the ‘stabilized temperature’ procedure), the sample is stabilized at a given temperature and the rebar is pulled-out at a constant displacement rate. For each procedure, by repeating at least 20 tests at different applied loads or at different stabilized temperatures, the $f_b-\theta$ relationship is determined. Figure 2 presents the oven and bond (TC1 and TC2) temperature evolutions (main Y-axis) in addition to the bond stress evolutions (secondary Y-axis) during a constant load test and a stabilized temperature test using the same mortar.

![Figure 2: Temperature and bond stress evolutions during a constant load test (left) and a stabilized temperature test (right)](image)

For the test carried out at constant load, a high difference between the oven temperature (measured near the heating resistances) and the bond temperature is observed due to the thermal inertia of concrete protecting the bond. Furthermore, the gap between the oven temperature and bond temperatures is highly linked to the oven properties and thermal exchange flux operating inside the oven. For this reason, the guide does not impose any criteria on the furnace properties but focuses on the bond temperatures. To frame the thermal kinetics, a criterion is imposed on a maximal test time of 3 hours, in order to remain representative of a fire duration. Between 80°C and 120°C, variation of the TC1 and TC2 temperatures can be observed leading to reversing the thermal gradient along the bond. This phenomenon is attributed to water vaporization that occurs in the porous network of the concrete material\(^7\). Thermal energy is consumed for phase transformation of water from liquid to gas, which opposes the temperature increase. The vaporization temperature depends on the pore pressure which is different near the concrete surface and the core of the cylinder\(^8\). This causes vaporization to occur at different temperatures and with different kinetics in the depth of the cylinder which disturbs thermal uniformity along the bond line. The phenomenon is linked to concrete properties (water content, permeability, porosity) and cannot be mastered technically. For this reason, no criterion on bond temperature control is attributed between 80°C and 120°C.
For the test performed at stabilized temperature, the oven and bond temperatures converge towards a thermal equilibrium after a long enough time. For this reason, this test procedure requires higher durations. A displacement controlled pullout test (in accordance with EAD330232-00-0601) allows to measure the slip of the rebar. The displacement measurement is not possible for a load controlled test since the temperature increase induces thermal displacements in addition to slip while also changing sensor sensitivity (resulting in an unreliable measurement).

Figure 3 presents the $f_b-\theta$ relationship obtained using three test protocols on the same vinyl ester mortar: i) at stabilized temperature (using an electric incubator), ii) at constant load (using a gas furnace), iii) at constant load (using an electric oven). The gas furnace was used to achieve a standard ISO 834-1 heating curve (EN 1991-1-2, section 3) conventionally used to assess structural fire resistance. This curve presents an exponential temperature increase in the furnace (around 300°C/min at the beginning) and yielded bond failures in less than 30 minutes. The electric heating followed a rate of 10°C/min near the oven resistances yielding failures between 30 minutes and 3 hours (depending on the applied load).

![Figure 3: Bond resistance versus temperature ($f_b-\theta$) relationship obtained for different test procedures on a vinyl ester mortar](image)

For most tests, the bond resistances obtained with gas heating appear similar or slightly lower than the ones obtained with electric heating. This difference is however slim in comparison to the gap existing between stabilized temperature tests (in red) and the constant load tests (in green). At 50°C the bond resistances vary by a factor of two between both test procedures. This has been observed for different mortar chemistries\(^2\). It is interpreted by 3 physical phenomena.

- First, the longer duration of the stabilized temperature test yield a higher post-curing level of the polymeric adhesive. During injection of the adhesive, both mortar components react together to achieve a three dimensional cross-linked network. At ambient temperature the reaction is never complete. By heating the material, molecular mobility is restored to unreacted polymeric groups which increases the degree of the reaction\(^11,12\). This results in a densification of the adhesive structure generally yielding higher mechanical properties.
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(depending on the chemistry). The longer thermal exposure in a stabilized temperature test favors strengthening of the material through postcuring.

- Secondly, the sustained load applied during a constant load test tends to lower the bond resistance. In fact, studies have shown that creep behavior is highly increased at higher temperatures.

- Finally, in a constant load test, hydric migration through the concrete sample exposed to a transient thermal increase (therefore presenting a radial thermal gradient) may induce the presence of moisture along the bond line while the load is applied. Studies have shown that, depending on the adhesive chemistry, water may decrease mechanical properties of the polymer.\textsuperscript{13,14,2}

Due to its representativeness (constant load in a structure in fire) and its conservative data (lower resistances), the constant load procedure is imposed in the guide for bond characterization. For technical and economic feasibility, electric heating was allowed as an alternative to gas heating (as long as the test duration is limited to 3 hours).

2.3 Position of the data points

The EAD\textsuperscript{4} determines some criteria on the applied load values of at least 20 tests in order to establish the $f_0$-$\theta$ relationship. The bond failure temperature is determined conservatively (to take into account the presence of a thermal gradient) with a weighted average calculated as $1/3$ of the higher measured temperature and $2/3$ of the lower measured temperature value between TC1 and TC2. The bond resistance is directly determined from the load assuming a uniform bond stress distribution (which has been assessed at 20°C and higher temperature in previous studies).\textsuperscript{15}

![Figure 4: Data points positioning and temperature reduction factor](image)

To achieve a descriptive distribution of data points, gaps between two neighboring data points shall not exceed 1 N/mm² vertically (controlled directly by the choice of the load) and 50°C horizontally. This second criterion depends directly on the failure temperature (not controlled) and is usually harder to achieve at low loads due to the convex (quasi-horizontal) trend of the curve. For this
reason, a higher concentration of tests may be required at low load levels. Furthermore, for conservative reasons, no extrapolations may be performed beyond the tested temperatures. In fact, the maximal temperature is determined as the average of the three highest temperatures to strengthen the representativeness of the mortar behavior. Only one or more data points are required above 10 N/mm² since this part of the curve (at low temperatures) is not taken into account to determine the reduction factor $k$ (as described in part 3.2).

3 Evaluation Method

3.1 Analytical $f_b$-$\Theta$ description

From the acquired data points (Figure 4), the $f_b$-$\Theta$ convex decreasing relationship is analytically described using an exponential trend. For some mortars, other trends present better fits while remaining simple (2 parameters) such as the power trend and the part wise segment trend. Nevertheless, the exponential description was imposed in the guide because it cannot yield negative bond resistances (unlike the power trend) and it imposes a repeatable fitting from one evaluator to the other (unlike the part wise segment description for which the segment lengths need to be defined).

3.2 Temperature reduction factor

Using the same philosophy as in the Eurocode 2\textsuperscript{16}, part 1-2, section 4 to describe the decay of material properties with temperature, the analytical curve of the $f_b$-$\Theta$ relationship is expressed as a temperature reduction factor $k$ equal to 1 at 20°C and which decreases with temperature (Figure 4). This reduction factor is calculated by making the ratio of the bond resistance obtained at elevated temperature with the required bond resistance as determined according to EAD330087-00-601\textsuperscript{4} (table 1).

\begin{center}
\begin{tabular}{|c|c|c|}
\hline
Concrete strength class & Design values of the ultimate bond resistance according to EC2 for good bond conditions & Required bond resistance for post-installed rebars \\
\hline
C12/15 & 1.6  & 7.1  \\
C16/20 & 2.0  & 8.6  \\
\textbf{C20/25} & \textbf{2.3} & \textbf{10.0}  \\
C25/30 & 2.7  & 11.6 \\
C30/37 & 3.0  & 13.1 \\
C35/45 & 3.4  & 14.5 \\
C40/50 & 3.7  & 15.9 \\
C45/55 & 4.0  & 17.2 \\
\textbf{C50/60} & \textbf{4.3} & \textbf{18.4} \\
\hline
\end{tabular}
\end{center}

Table 1: Required bond resistance (from TR023\textsuperscript{1})

The required bond resistance partially originates from an empirical study\textsuperscript{17,18} on cast-in rebars in uncracked confined concrete. For a sufficient concrete edge distance, necessary to avoid splitting failure, the experimental average bond resistance in C20/25 concrete is equal to 10 N/mm². The
design bond resistance of CIR covered in EN 1992-1-1\textsuperscript{5}, section 8 is equal to 2.3 N/mm\textsuperscript{2}. It is safety considered that PIR cannot present higher resistances than CIR. The value of 10 N/mm\textsuperscript{2} is therefore considered as a required threshold necessary to guarantee that a PIR can be qualified using a bond resistance of 2.3 N/mm\textsuperscript{2}. With the assumption that this equivalence is applicable at high temperature for each concrete class, the k factor is limited to the value 1 at low temperatures up to $\theta_k$, (as long as the bond resistances are higher than 10 N/mm\textsuperscript{2} for C20/25 concrete). For mortars that present a bond resistance lower to the required value at 20°C, a proportional reduction is directly applied to determine the k factor. At temperatures higher than $\theta_k$, the k factor decreases proportionally to the $f_b$ analytical curve. At temperatures higher than $\theta_{max}$, the k factor is conservatively equal to 0 since no extrapolation is allowed.

4 Design Method

4.1 Bond design resistance at ambient temperature

The EAD330087-00-601\textsuperscript{4} provides for each concrete class a given design bond resistance (referenced $f_{bd,20^\circ C}$ to differentiate it with the fire design bond resistance $f_{bd,fire}$). The characteristic bond resistance $f_{bk}$ can be determined from the 20°C design bond resistance $f_{bd,20^\circ C}$ by taking into account the material safety factor associated to concrete $\gamma_{M,20^\circ C}=1.5$ (EN 1992-1-1\textsuperscript{5}, section 2). With the same reasoning, the reference fire design resistance $f_{bd,fire}(20^\circ C)$ can be determined from the characteristic bond resistance $f_{bk}$ by taking into account a material safety factor in a fire situation $\gamma_{M,fire}=1.0$. Finally, the fire design bond resistance as function of temperature $f_{bd,fire}(\theta)$ can be determined by considering the k reduction factor (equation 1). It can be noted that for CIR, since the design bond resistance is directly proportional to the concrete tensile strength, the decay with temperature is also expressed with a k reduction factor associated to concrete.

$$f_{bd,20^\circ C} = \frac{f_{bk}}{\gamma_{M,20^\circ C}}$$
$$f_{bd,fire}(20^\circ C) = \frac{f_{bk}}{\gamma_{M,fire}}$$
$$f_{bd,fire}(\theta) = f_{bd,fire}(20^\circ C).k(\theta) = f_{bd,20^\circ C}.\frac{\gamma_{M,20^\circ C}}{\gamma_{M,fire}}.k(\theta) \quad (1)$$

Where:
- $f_{bk}$ is the product dependent characteristic value
- $f_{bd,20^\circ C}$ is the design bond resistance at 20°C (equal to 2.3 N/mm\textsuperscript{2} for C20/25 concrete)
- $f_{bd,fire}(20^\circ C)$ is the fire design bond resistance at 20°C (equal to 3.5 N/mm\textsuperscript{2} for C20/25 concrete)
- $f_{bd,fire}(\theta)$ is the fire design bond resistance that depends on temperature
- $\gamma_{M,20^\circ C}$ is the material coefficient at ambient temperature (equal to 1.5)
- $\gamma_{M,fire}$ is the material coefficient in a fire situation (equal to 1)
- $k(\theta)$ is the temperature reduction factor that depends on temperature

At 20°C, the k factor is equal to 1.0. Thus, the fire design resistance $f_{bd,fire}$ is equal to 3.5 N/mm\textsuperscript{2} while the ambient temperature design resistance $f_{bd,20^\circ C}$ is equal to 2.3 N/mm\textsuperscript{2}. The fact that the fire design resistance is higher than the cold design resistance at 20°C (in other words, at the very beginning of a fire), may appear counter-intuitive but is purely due to the safety coefficients. This
mismatch at 20°C is overpassed by the fact that ambient temperature design shall always be performed before fire design. Hence, designing a bond geometry at ambient temperature will already offer a portion of fire safety (quantified in a duration of fire resistance).

The design bond resistances from EAD330087-00-6014 for each concrete class directly originate from the design bond resistances of cast-in rebars covered in Eurocode 25. This limitation implicitly relies on the conservative assumption that a PIR cannot present higher bond resistances than a CIR. The progress that has occurred in enhancing the performances of structural adhesive technology is therefore not taken into account today with the current design methods.

4.2 Temperature distributions

The determination of a load resisting capacity using the bond resistance requires the knowledge of the thermal distribution along the bond at a given time during the fire. Thermal fields are determined by using the method described in EN 1991-1-29, section 3 generally with a finite element software. This method requires as entry data: i) a fire time-temperature relationship, ii) exchange coefficients to determine the thermal flux, iii) conductive properties of concrete, iv) a structural geometry.

i. The standard ISO 834-1 time-temperature relationship (EN 1991-1-29, section 3) is used to describe the gas temperatures of the fire.

ii. The emissivity and convective exchange coefficients (are provided by EN 1991-1-29, appendix A) are used to determine respectively the radiative and convective thermal flux applied to the concrete surfaces exposed to fire.

iii. The concrete thermal properties (variations of thermal conductivity, mass density and specific heat) are provided by EN 1992-1-29, section 3. Concrete type and moisture content influence the thermal diffusivity. Conservative thermal fields can be established by considering the peak of the specific heat associated to a concrete water percentage of 1.5%.

iv. The structural geometry directly influences thermal fields by the area and position of the fire exposed surfaces of concrete in addition to the concrete cover thermally protecting the bond.

Figure 5 presents an example of calculated temperatures in the case of a slab exposed to fire on the lower surface. Although finite elements allow the determination of thermal fields for different geometries, some thermal distributions associated to simple geometries are sometimes presented in abacus (EN 1992-1-216, appendix A).
4.3 Determination of the resisting load

The design load resistance capacity is calculated by integration of the bond resistances $f_{bd,fire}(\theta)$ along the lateral surface of the rebar (equation 2). Since the temperature profiles are not expressed analytically, the integration is carried out numerically and consists in adding the bond resistances on short bar segments each presenting a different temperature (equation 3).

$$N_{Rd,fire} = \pi \cdot d \cdot \int_0^l f_{bd,fire}(\theta(x)) \cdot dx = \pi \cdot d \cdot f_{bd,20^\circ C} \cdot \gamma_{M,20^\circ C} \cdot k(\theta(x)) \cdot dx \quad (2)$$

$$N_{Rd,fire} \approx \pi \cdot d \cdot f_{bd,20^\circ C} \cdot \gamma_{M,20^\circ C} \cdot \gamma_{M,fire} \cdot \sum_{i=0}^{l_v} k(\theta_i) \cdot \Delta x \quad (3)$$

Where:
- $N_{Rd,fire}$ is the design load resistance at a given time during the fire
- $f_{bd,20^\circ C}$ is the design bond resistance at $20^\circ C$ (equal to 2.3 N/mm² for C20/25 concrete)
- $\gamma_{M,20^\circ C}$ is the material coefficient at ambient temperature (equal to 1.5)
- $\gamma_{M,fire}$ is the material coefficient in a fire situation (equal to 1)
- $k(\theta)$ is the temperature reduction factor that depends on temperature
- $l_v$ is the embedment depth of the bonded rebar

For representativeness, the segment length is lower than 2 times the bar diameter and is generally taken around 10 mm. For safety, the maximal temperature of each segment is considered in order to underestimate the bond resistance along the segment.

Figure 6 presents an example of how the load resistance is determined from the bond resistances associated to a temperature profile. The safety of this method can be questioned by the fact that it does not take into account displacement compatibility. The integration assumes that, at failure, all segments each present the highest bond stress (that can be reached at a given temperature) regardless of the slip profile. This hypothesis nevertheless appears sufficiently representative through experimental and theoretical support to predict bond failure at high temperatures.

Figure 5: Variation of temperatures with concrete cover at different fire durations in the case of a slab exposed to fire on the lower surface
4.4 Structural configurations and output

Bond resistance values are generally communicated to the manufacturer depending on the structural configuration which influences temperature fields. Two main structural configurations, presented in Figure 7, are usually studied associated to the rebar applications (overlap joint or anchorage). Thermal boundary conditions are generally considered by simulating a fire in a room. This leads to exposing the lower surface of the slab to fire or exposing the lower surface and the lateral sides of a beam.

For a slab-slab or beam-beam configuration, the temperature is uniform along the bond since the isotherms are parallel to the fire exposed concrete surfaces. The design bond resistances (depending only on the fire duration and the concrete cover) are therefore directly communicated. From these values, the load capacities can be calculated by the design office by implementing the bond geometry.
For a slab-wall or beam-wall configuration, a thermal gradient along the bond requires performing the integration including the bond geometry. Load resistances are therefore communicated depending on the fire duration, concrete cover, rebar diameter and bond length.

5 Conclusion

The fire evaluation is performed through experimental bond characterization at high temperature. The product dependent $f_b-\theta$ relationship is converted into a temperature reduction factor $k$. Determination of the design load resistances is established by integration along the bond length using the temperature distributions.

The method presents two major sources of safety. First, during the determination of the $k$ factor, the limitation is applied to a required bond resistance (such as 10 N/mm² for C20/25 concrete). This limitation, in accordance with design at 20°C (EAD330087-00-601^4) takes into account the equivalence between experimental and design bond resistances. Secondly, a limitation of the design bond resistance at 20°C (such as 2.3 N/mm² for C20/25) is issued from design at ambient temperature (in accordance with Eurocode 2^5). These two conservative statements yield a safety reduction of a ratio higher than 10 on the experimental bond resistance for many structural adhesives manufactured today. As a result product performance at low temperatures is not captured in the current design method.

It is possible to increase the representativeness of the mortar description at high temperature expressed through the $k$ reduction factor by performing tests at higher temperature (experimentally) or improving the analytical trend curve (by function fitting). However, these features present limited impact on describing the performances of a product in comparison to the two safety sources originating from the harmonization of fire with ambient temperature design. The modification of fire design resistance values relies mostly on the possibility to improve the method associated to cold design through enhanced knowledge on the mechanical behavior of PIRs.

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A NUMERICAL METHOD TO EVALUATE THE PULL-OUT STRENGTH OF BONDED ANCHORS UNDER FIRE

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ABSTRACT

The bond strength of bonded anchors is known to be strongly dependent on temperature and it reduces rapidly with rising temperature. The rate of degradation in bond strength depends on the adhesive material (inorganic/organic/vinylester/epoxy) used and hence is product dependent. A bonded anchor under fire in tension can have various failure modes viz., steel failure, pull-out failure and concrete cone failure. At present EOTA TR 020 (2004) can be used for evaluating only steel failure under fire for bonded anchors. However, due to the fact that pull out failure of the anchor might be decisive, it is recommended to evaluate the bonded anchors by performing fire tests. Evaluation by fire tests has the limitation that the anchor can be qualified only for a particular fire scenario and its fire rating can be significantly different under different fire scenarios.

The paper presents a numerical model for computing the pull out strength of bonded anchor in reinforced concrete subjected to generalized fire loads. The model involves solving 2D transient heat transfer equations to obtain the temporal and spatial distribution of temperature, as the first step. The transient heat transfer analysis considers thermal properties of concrete and steel as a function of temperature. Once, the temperature variation along the anchor depth is obtained, the variation of bond strength along the anchor length is computed. The pull out capacity of the anchor is then calculated by integrating the temperature dependent bond strength over the embedment depth of the anchor. The model is validated against the experiments available in literature and a good comparison is shown between experimentally obtained and numerically calculated values of the pull-out strength. Furthermore, the paper presents a study on the pull-out strength of bonded anchors under different fire scenario.

1 Introduction

With the increasing use of anchors in structural engineering, the behaviour of anchors under accidental situations such as fire has gained relevance.1,2 In the event of fire a structure is exposed to high temperature within a short period of time and thus, induces temperature gradients in the structural member. The rise in temperature leads to degradation in material properties of both concrete3,4 and steel5. Due to this degradation in material properties and cracking, the load carrying capacity of an anchor under fire is reduced. This reduction in capacity is more rapid in case of bonded anchor systems because of the use of the polymeric mortar, which can cause safety issues.
The polymeric mortar between concrete and steel is responsible for the rapid loss of the anchor capacity because the properties of the polymeric adhesive material changes significantly over a short temperature range. The effect of temperature on polymeric materials is quantified in terms of glass transition temperature (Tg). It is defined as the temperature above which the adhesive material shows a transition from glassy behaviour to rubbery behaviour.

At present EOTA TR 020 (2004) can be used for evaluating only steel failure under fire for bonded anchors, in general. The guidelines can also be used for evaluating other failure modes (pull-out & concrete cone) but only for bonded expansion and bonded undercut anchors with an approval for cracked and non-cracked concrete. However, it also recognises the fact that pull-out failure may be decisive due to reduction of strength of the bond material and hence, it is recommended to evaluate the bonded anchors by performing fire tests.

Evaluation of bonded anchors by fire tests has the limitation that the anchor can be qualified only for a particular standard fire scenario. However, its fire rating can be significantly different under different fire scenarios. This paper presents a method to evaluate the pull-out capacity of the bonded anchors subjected to generalized fire loads.

2 Literature review

At present limited data is available on bonded/adhesive anchors exposed to fire. The available research in the field of anchorages in more focused on the performance of post installed rebars at elevated temperatures.

Experimental studies were conducted by Paterson (1978) with expansion, cast-in and bonded anchors exposed to standard fire. The bonded anchor system consisted of M10 anchor rod with 75mm effective depth and was installed using polyester base adhesive. The anchors failed in pullout within 10-15min of fire with an anchor temperature between 330-440 °C. It was highlighted by Paterson (1978) that anchors/fastenings may fall earlier in a fire than the structural element itself, on which the anchor is installed. Thus, endangering the lifes of occupants & fire fighters and can cause blockage of escape routes.

Bergmeister & Rieder (2001) and Lubloy & Balazs (2007) conducted experiments to study the effect of temperature on expansion anchors and bonded anchors. But their studies were focused on the residual capacity of anchors i.e., in post fire scenario.

Pinoteau et al (2012) conducted an experimental investigation on the mechanical properties of three different polymer resins with temperature. The three types of resins studied included epoxy, vinylester and urethane methacrylate. They conducted dynamic mechanical thermal analysis to determine the glass transition temperature (Tg). It was found that Tg values may change substantially depending on the resin and the determination method.

Zhang et al (2016) conducted pull out test using steel sleeve method as per Chinese standard to determine the effect of temperature on bond stress-slip relationships for adhesive anchors. They conducted tests using three commonly used organic anchorage adhesive (one was vinyl resin based; and two were epoxy based). The results show a strong dependency of bond strength on increasing
temperature. The bond strengths were around or lower than 10% at temperatures exceeding 260 °C and the vinylester adhesive was found to be more sensitive than epoxy adhesive to temperatures.

Recently some numerical studies were done by Patil and Thiele (2015)\textsuperscript{12} to study the factors affecting the temperature distribution along the anchor depth. The effect of humidity in concrete, anchor rod diameter and mortar layer was studied. Their study showed that when the anchor is directly exposed to fire the predicted anchor temperatures are higher when the mortar layer is modelled because it acts as an insulating material. But, the conclusion is valid under the assumption of constant thermal properties for mortar layer.

3 Numerical model

The model consists of carrying out a 2D transient heat transfer equations to obtain the temporal and spatial distribution of temperature, as the first step. Thereafter, the pull-out capacity of the anchor is computed by integrating the temperature dependent bond strength over the embedment depth of the anchor. For carrying out the analysis the domain which consists of concrete & steel is divided into segments along its width and depth. Using lumped system concept, which is common approximation in transient conduction conduction problems\textsuperscript{13}, each segment is considered to have a uniform temperature and properties, lumped at the center of that segment. All computations are done at the centre of the sector, which represent the complete sector. Temperature dependent thermal properties (conductivity and specific heat) of concrete and steel were used. The presented model has been numerically implemented using an inhouse code.

The numerical model presented in this paper is based on the following assumptions:

1. Effect of spalling has been ignored. Hence, the model is valid in general for normal strength concrete where the chances of spalling are small due to higher porosity.
2. Constant temperature field has been assumed along the length of the structural member in which the anchor is installed. This assumption reduces the heat transfer analysis to 2D.
3. Uniform bond stress is assumed along the anchor embedment depth.

3.1 Heat transfer analysis

In the field of structural fire safety, estimating the temperature distribution inside the members is the first and the most important step. In an event of fire the heat transfer from the hot gases to structure takes place though convection and radiation at the boundaries of the member, and then the heat propagates into the member through conduction.

The governing differential equation for 2D transient heat conduction problem is given by Eq. (1). The governing equation is solved using implicit Central Difference Scheme and iterative solver to obtain the spatial and temporal distribution of temperature, $T(x,t)$. Eq. (2) states the Neumann boundary condition that needs to be satisfied.

\[
\rho c \frac{\partial T}{\partial t} = k \left[ \frac{\partial^2 T}{\partial x^2} + \frac{\partial^2 T}{\partial y^2} \right]
\]
\[-k \frac{\partial T}{\partial n} = h[T_g - T_s] + \varepsilon \sigma [(T_g + 273)^4 - (T_s + 273)^4]\]  \hspace{1cm} (2)

Where:
- \(k\) is the thermal conductivity (W/m °C),
- \(\rho\) is the mass density (kg/m³),
- \(c\) is the specific heat of solid (J/kg °C),
- \(h\) is convective heat transfer coefficient (25 W/m² °C),
- \(\varepsilon\) is Stephen Boltzmann constant \((5.667 \times 10^{-8} \text{ W/m}^2 \text{ °K}^4)\),
- \(\sigma\) is surface emissivity (0.8),
- \(T_g\) is gas temperature (°C),
- \(T_s\) is surface temperature (°C).

The thermal properties used viz., thermal conductivity & specific heat as function of temperature for concrete and steel are shown in Figure 1 (a) and (b) respectively.

![Figure 1: Variation of thermal properties with temperature](image)

### 3.2 Pull out capacity

Based on the transient heat transfer analysis carried out in the first step, the temperature variation along the anchor depth is known. The second input required for this step is the variation of bond strength with increasing temperatures. This variation can be obtained for each product according to the European Technical Document\(^{14}\). Using the above stated inputs (temperature along anchor depth & temperature dependent bond strength) the variation of bond strength along the anchor depth is known. The pull out capacity of the bonded anchor is computed by integrating the temperature dependent bond strength over the anchor length. The numerical integration is carried out using Simpson’s rule, stated in Eq. (3). It is based on quadratic interpolation between two points ‘a’ and ‘b’ and gives accurate results for polynomial function \(f(x)\) of degree three or less\(^{15}\).

\[\int_a^b f(x)dx \approx \frac{b-a}{6} \left[ f(a) + 4 f \left( \frac{a+b}{2} \right) + f(b) \right]\]  \hspace{1cm} (3)

Where:
- \(a, b\) implies integration limits
- \(f(x)\) is the function to be integrated
\[ f(a) \] is value of the function at point ‘\( a \)’
\[ f(b) \] is value of the function at point ‘\( b \)’
\[ f((a+b)/2) \] is value of the function at point ‘\((a+b)/2\)’

\section{4 Validation}

To establish the numerical validation of the inhouse code, experiments performed by Muciaccia et al (2016)\textsuperscript{16} were selected. Their experimental study aimed to investigate the post installed connections using vinylester polymer under high temperatures. They performed 6 tests at steady state (heating to steady state before loading) and 14 tests under transient conditions (loaded specimen, heated to failure). Tests performed under transient conditions were picked for validation. The specimen consists of a 12mm ribbed bar post installed (using vinylester polymer mortar) in a steel cased C20/25 uncracked concrete cylinder with diameter 150mm and 200mm height, as shown in Figure 2. The bar was loaded with different sustained loads before heating the specimen at a rate of approximately 10°C/min until failure. The load was maintained constant during the heating period. The specimen was exposed to fire from all sides except the side on which anchor was installed.

Figure 2: Specimen used by Muciaccia et al (2016)\textsuperscript{16}

Figure 3 shows the experimentally\textsuperscript{16} obtained variation of bond strength with temperature and the corresponding fitted exponential curve used in the numerical model. Comparison between the numerically predicted temperature and experimental observation as shown in Figure 4, shows a good agreement between the two. Figure 5 shows the comparison between the numerically predicted and experimentally obtained variation of pull-out capacity with exposure time. The overpredicted between the time 50 – 100 min is because of the under predicted temperatures in that time range. This can be attributed to the possible experimental scatter or to possible scatter in the heating rates for different specimens.

Figure 3: Experimental and fitted variation of bond strength with temperature
Further validation of the proposed model was made against the experiments on bonded anchors exposed to standard fire, reported by Patil and Thiele (2015)\textsuperscript{12}. The anchor system consisted of M12 anchor rod with an embedment depth of 110mm. The relation between the bond strength and increasing temperature for the mortar used is shown in Figure 6 and the same was used in the numerical model. In the numerical model a loading fixture of 90mm width as per EOTA TR 20\textsuperscript{7} was assumed. Based on the experimental observations of Paterson (1978)\textsuperscript{8} where the anchor failed within 10-15 min, when directly exposed to fire, it was assumed that the anchor rod was insulated.
As shown in Figure 7 (a) and (b), a large difference can be observed in the temperature variation along anchor depth for the cases when the anchor itself is exposed to fire (acts as a heat transfer path) and when insulated (along with fixture). It can be concluded, that for the temperature variations shown in Figure 7 (a) and the bond strength degradation shown in Figure 6, the anchor would fail in pullout between 30 -60 min.

![Graph](image1.png)

(a) Case: Anchor directly exposed to fire

![Graph](image2.png)

(b) Case: Insulated anchor & fixture

Figure 7: Temperature variation along the anchor depth

(Distance on y-axis implies the anchor depth; 0 = point on the exposed surface)

The pullout capacity was calculated using the computed temperatures (Figure 7 (b)) and the variation of bond strength with temperature (Ref. Figure 6). One of the missing input parameter was $O_{\text{max}}$, i.e., the temperature above which the bond strength shall be considered to be zero. Hence, two values were chosen for the validation; 220 °C (as could be seen from Figure 6) and 250 °C. The comparison between the experimental pullout capacities and the computed capacities are shown in Figure 8. The predictions made using the presented model are reasonably good based on the input parameters available.

![Graph](image3.png)

Figure 8: Comparison of predicted and experimental anchor load carrying capacity under standard fire scenario
5 Parametric study

After validating the proposed numerical method, a parametric study has been presented to investigate the effect of different fire exposure and insulating the anchor/fixture. Table 1 summarises the various cases for the parametric study. For the parametric study: the thermal properties for concrete and steel were the same as used for the validation; the degradation of bond strength with temperature was assumed to follow the law as per Eq. (4); M12 anchor with an embedment depth of 120 mm was assumed.

\[
f_{bm}(\theta) = f_{bm} (a \cdot e^{-b \cdot \theta}) \quad \quad \quad \quad \quad \theta_{min} \leq \theta \leq \theta_{max}
\]

Where,
\[
\theta = \text{temperature (°C)}
\]
\[
f_{bm}(\theta) = \text{mean bond resistance at temperature } \theta \text{ (MPa)}
\]
\[
f_{bm} = \text{mean bond resistance at ambient conditions (12 MPa)}
\]
\[
a, b = \text{exponential fitting curve constants (a = 1,3091; b = 0,008)}
\]
\[
\theta_{min} = \text{minimum temperature required to initiate the degradation in bond strength (40 °C)}
\]
\[
\theta_{max} = \text{temperature above which the bond resistance is zero (338 °C)}
\]

Table 1: Various cases for parametric studies

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Fire Exposure (Ref. Figure 9)</th>
<th>Anchor/Fixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>DF-I</td>
<td>Design Fire (DF) (1 hr Heating as per ISO 834; cooling at 500 °C/hr)</td>
<td>No Fixture Unprotected Anchor</td>
</tr>
<tr>
<td>DF-II</td>
<td>Design Fire (DF) (1 hr Heating as per ISO 834; cooling at 500 °C/hr)</td>
<td>Fixture 90 mm (width) Protected anchor &amp; fixture</td>
</tr>
<tr>
<td>HF-I</td>
<td>Hydrocarbon Fire (HF) (As per Eurocode1)</td>
<td>No Fixture Unprotected Anchor</td>
</tr>
<tr>
<td>HF-II</td>
<td>Hydrocarbon Fire (HF) (As per Eurocode1)</td>
<td>Fixture 90 mm (width) Protected anchor &amp; fixture</td>
</tr>
</tbody>
</table>

Figure 9: Temperature-time relationships for different fire exposure

The fire scenarios were so chosen to investigate the effect of fire which is more severe than the standard fire exposure. Hence, hydrocarbon fire as per Eurocode1 was selected. The other chosen
fire scenario is the design fire where the heating follows the ISO834 curve for 1 hr followed by cooling at the rate of 500 °C/hr. The purpose of design fire is to emphasis on the fact that the anchor may also fail during the cooling phase of real/design fire.

Figure 10 shows the temperature contour for the case DF-I. It can be seen that due to its high conductivity anchor acts as a heat path. It leads to reduced temperature of concrete near the surface and increase in temperature along the anchor depth. It should also be noted that because of the high thermal inertia of concrete, the temperatures inside the concrete are increasing even during the cooling phase of the fire. Figure 11 shows the variation of the temperature and the corresponding mean bond strengths along the anchor depth. It can be observed that the effective embedment depth of the anchor continues to reduce even during the cooling phase of fire.

![Temperature contours for Case: DF-I](image)

Figure 10: Temperature contours for Case: DF-I

![Temperature and bond stress variation along the anchor depth for Case: DF-I](image)

(a) 30 min  (b) 60 min  (c) 90 min

Figure 11: Temperature and bond stress variation along the anchor depth for Case: DF-I

(—— = Temperature; ——— = Bond strength)

Figure 12 shows the temperature contours for case DF-II. Since, the fixture and the anchor rods are assumed to be insulated, the heat transfer to the anchor takes place from the surrounding concrete. It can be observed in this case also that the anchor rod serves as a heat path and the anchor temperatures continues to increase during the cooling phase of fire (Ref. Figure 13).
The geometric details for the cases HF-I & HF-II are the same as for cases DF-I & DF-II respectively. These cases are analysed for hydrocarbon fire scenario, with a convective heat transfer coefficient of 50 W/m² K as compared to a value of 25 W/m² K for design fire. Finally, the pullout capacities were evaluated (using the presented numerical model) for all the four cases under study and are plotted in Figure 14.

It can be observed for case DF-I that the pull-out capacity of anchor reduces to zero at \( \approx 90 \text{min} \), i.e, 30 minutes into the cooling phase. On comparing cases DF-I & HF-I, a reduction in failure time
(zero pull-out capacity) from ≈ 90min to ≈ 45min is observed. In case of protected anchor under design fire (Case: DF-II), the rate of degradation of pullout capacity reduces during the cooling phase but the pullout capacity continues to reduce and reaches a saturation value. The degradation in pullout capacity in case HF-II is higher as compared to DF-II, as expected.

6 Concluding remarks

A numerical model for computing the pull-out capacity of bonded anchor in reinforced concrete subjected to generalized fire loads has been presented. The model involves solving transient heat transfer equations to obtain the temporal and spatial distribution of temperature, as the first step. The output from first step provides the temperature variation along the anchor depth. Hence, the variation of bond strength along the anchor length can be computed. The pull out capacity of the anchor is then calculated by integrating the temperature dependent bond strength over the embedment depth of the anchor. The proposed approach does not account for the spalling of concrete due to fire. Hence, it is applicable to situations where the spalling due to fire can be ignored. The model is validated against the experiments available in literature and a good comparison is shown between experimentally obtained and numerically calculated values of the pull-out strength.

A parametric study has also been presented on the pullout strength of bonded anchors under Design Fire (DF) and Hydrocarbon Fire (HF) scenarios. The degradation of pullout capacity for unprotected and protected anchors under the two fire scenarios (DF & HF) has been discussed. The pullout capacity reduces drastically in case of unprotected anchors (without fixture), and reduces faster for severe fire scenarios like hydrocarbon fires. Insulating the anchor/fixture can delay the pullout failure time of bonded anchors but it is important to account for the thermal inertia effect of concrete to ensure that the anchor does not fail during the complete fire duration.

It was observed that the degradation of pullout capacity of the bonded anchor depends on the fire scenario under consideration. Hence, the anchors that survived/did not fail under standard fire may not show the same failure time under real fire scenarios (Design Fires). It has been demonstrated that the anchor may also fail during the cooling phase of a real fire. Thus, highlight the need for methods to evaluate the pullout failure of bonded anchors under generalised fire loads.

References:


4. ACI 318-14 , "Building code requirements for reinforced concrete" Detroit, Michigan: American Concrete Institute, 2014.


QUALIFICATION OF BONDED ANCHORS IN CASE OF FIRE

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ABSTRACT

In the few last years, the number of technical approvals for chemical anchor systems has increased significantly. Part five of the European technical guideline 001¹ provides information for the evaluation of chemical anchor systems for cold-state design. For the qualification of fastenings under fire exposure, it is state of the art to execute fire tests according to the technical report TR 020². For bonded anchors with flexible embedment depth and different anchor diameters, the implementation of this guideline leads to a huge number of expensive fire tests. Besides that, the information in TR020 does not cover pull-out failure of bonded anchors.

The aim of the research project “bonded anchors in case of fire”, supported by Deutsches Institut für Betontechnik (DIBt) and industrial partners, is to get a better understanding for the load-bearing behaviour of chemical anchor systems under fire exposure. Therefore, a combination of different types of fire tests and thermal simulations with finite element analysis have been applied. In a first step, the temperatures along the anchor during the fire exposure were simulated and compared with temperature measurements. The thermal simulations were in a good accordance with the results of temperature measurement, also with a varying in parameters like embedment depth and anchor diameter. Afterwards, the relation between bond strength and rising temperature was analysed. Therefore, the fire test for post-installed rebar connections with mortar according the corresponding European Assessment Document has been applied. The integration of bond strengths correlating to the equivalent temperature along the embedment depth delivers the fire resistances. This design method allows to define fire resistances for any combination of anchor diameter and anchorage depth with a manageable number of fire tests.

1 Introduction

For anchorages, no DIN or rather EN – standards exist, therefore they have to be qualified by a general technical approval (allgemeine bauaufsichtliche Zulassung, abZ) or a European Technical Approval (ETA). For the several anchor types additional Technical Reports (TR) are available, which define special issues.

The assessment of the fire resistance of anchorages is defined in technical report TR 020². This report covers in particular the assessment of mechanical anchor systems. For chemical anchor systems only anchors with the failure mode steel failure are covered. The information about bond failure of chemical anchor systems in this document are insufficient.
A bonded anchor system consisting of an anchor rod and an injection mortar transfers loads by adhesive bond. For this anchorage, three main types of failure modes are possible: bond failure, steel failure and concrete cone failure. Under fire action, the failure probability for bond failure rises, because of the degradation of bond strength with higher temperatures. The same is valid for the failure mode steel failure, the steel strength also degrades highly for high temperatures. Because of the increase in failure probability for bond failure and steel failure, the probability for concrete cone failure for a single anchor decreases. In this research project the failure mode bond failure was investigated.

At the moment, no uniform standard for the evaluation and execution of fire tests on bonded anchors in case of fire exists. It is the aim of the research project “bonded anchors in case of fire” worked on by the Technical University of Kaiserslautern in collaboration with DIBt and several industrial partners, to understand the load bearing behaviour of bonded anchors in case of fire and determine an evaluation and testing procedure.

2 Bond failure

In general, the injection mortars used for chemical anchor systems are not made for the high temperatures appearing during fire exposure. With increasing temperature, the bond strength degrades. Especially for short embedment depth the probability of bond failure is high.

The evaluation concept explained below should give the opportunity to calculate the fire resistance loads for every combination of anchor diameter and anchorage depth of a chemical anchor system. Therefore, a combination of FE simulations and fire tests according to the EAD for post installed rebars were used. Figure 1 to Figure 4 exemplarily show the steps of fire resistance calculation. First of all, the time in the uniform temperature time curve (UTTC) for which the fire resistance load should be calculated, has to be defined. Afterwards the temperatures along the embedment depth of the chosen anchor (anchor diameter and anchorage depth) were simulated with the FE program ANSYS. More details about the simulation of temperatures along the embedment depth during fire are given in chapter 2.1. Parallel the reduction of bond strength corresponding to the increase of temperature was determined by using the fire test for post installed rebar connections according to EAD 330087-00-0601. More information about the determination of the bond strength-temperature relation is given in chapter 2.2. With the two relations defined before (temperature – embedment depth and bond strength – temperature) the reachable bond strength in the fire test can be calculated in sections along anchorage depth. Figure 4 shows the result of this calculation. The integral of this graph finally provides the calculated fire resistance load for the chosen time in the UTTC, anchor diameter and embedment depth. The single steps of the calculation are explained in detail in the chapters 2.1 to 2.3.
2.1  FE simulation – temperatures along embedment depth

The numerical FE-Simulation was executed with the simulation software ANSYS. With a thermal simulation, the temperature along the anchorage depth can be determined for every combination of anchor diameter and anchorage depth.

The temperature-dependent values of the material properties of steel and concrete (specific heat and thermal conductivity) given in Eurocodes DIN EN 1992-1-2\textsuperscript{5}, and DIN EN 1993-1-2\textsuperscript{6} were used. For the heat transfer coefficient between concrete and air a value of 25 W/mK was used. For the contact areas to concrete a value of 4 W/mK was taken. An emissivity of 0.7 for steel and concrete was used to take radiation into account. As a result of the simulation for each anchor the temperature along the embedment depth can be determined as shown in Figure 2.
Additionally to the parameters anchor diameter and embedment depth, the simulation considers the steel grade and fixture used in the tests. Figure 5 shows the geometry of test members in the simulation. Exemplary the analysis of the influence of anchorage depth is figured. The point of measurement for which the comparison is done, is marked with a red dot.

Figure 5: Geometry of test members by testing the influence of anchorage depth in simulation

The parameter study results in following conclusions:

- Short embedment depth results in higher temperatures at the same point of measurement
- Big anchor diameters result in higher temperatures at the same point of measurement
- Anchor rods made out of stainless steel result in lower temperatures at the same point of measurement
- The use of a fixture results in lower temperatures at the same point of measurement

Furthermore, the results of the simulation were compared to temperature measurements from real fire tests. As an example, Figure 6 shows the comparison between simulation results and fire test results for a M12 anchor with 60 mm anchorage depth made out of c-steel. It can be seen, that test results and simulation results are in a good accordance. But especially the temperatures measured at an anchor with fixture are less than the simulated values.
Figure 6: Comparison of test results with simulation results

For further information about the thermal simulation and the comparisons with real fire tests see the report of the research project “Verbunddübel im Brandfall”.

2.2 Bond strength – temperature relation

To use the results of the thermal simulation, a relation between temperature and bond strength is required. Therefore, the fire test described in the EAD 330087-00-0601 “systems for post-installed rebar connections with mortar” was used.

Figure 7: Test description for fire tests acc. to EAD 330087-00-0601

In the tests, steel coated concrete cylinders were used. A rebar has to be installed in the middle of the cylinder and has to be loaded with a sustained load $N_{\text{test}}$. During the anchor is loaded, a thermal load has to be applied on the lateral sides of the cylinder. The temperatures at the
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anchor were recorded at two points of measurement (TC1 and TC2). A weighted average value of the temperatures at TC1 and TC2 is given as a result by the time of failure of the anchor. The guideline provides a minimum number of 20 tests with different sustained loads $N_{\text{test}}$. The test program leads to a mortar dependent relation between temperature and bond strength.

For the use in the calculation of fire resistance loads of bonded anchors in case of fire, tests with threaded rods were executed. The results are shown in Figure 8. Besides that, the influencing factors of anchor type, anchor diameter and moisture of the concrete member were tested.

At the current state of research slightly higher failure temperatures can be reached with threaded rods. The anchor type in general has an influence especially on the failure temperatures for high bond strength (Failure temperatures near ambient temperature). For the parameters anchor diameter and moisture no significant influence can be determined.

![Comparison rebar1 threaded rod](image)

Figure 8: Comparison of tests with threaded rod and rebar

## 2.3 Calculation of fire resistance loads regarding bond failure

With the relations temperature – embedment depth and bond strength – temperature worked out before, each section of embedment depth a bond strength can be assigned. By integration of the bond strength – embedment depth graph (Figure 4), a calculation of fire resistance loads is possible.

For confirmation of this thesis a comparison between calculated values and results of real fire tests were done. An example for the comparison is given in Figure 9, which shows the results for a threaded rod with a diameter of 12 mm and an embedment depth of 70 mm made out of c-steel. The values for steel failure given in this figure are the average values of steel failure determined in the database described in chapter 3, for threaded rods M12.
Figure 9: Calculation of fire resistance loads

Figure 9 shows that the results of the calculation and the test results are in good accordance. Only for long fire resistance times (120 min) the calculation results in no fire resistance load for this anchor, while the test results show that small loads can be held. Further research will examine this problem.

3 Steel failure

If the fire resistance regarding bond failure is high enough, the steel strength of the anchor rod is decisive. The fire resistance of an anchor regarding steel failure is not dependent on the anchor type or mortar product, but on the steel grade of the anchor rod. However the technical report TR 020 demands on new fire tests for each product or the values given in the simplified design method can be used.

Experience showed that the execution of fire tests results in much higher fire resistance loads as given in the simplified design model. A database with results of fire tests with steel failure from different approval procedures shall verify this assumption. Steel failure can be split in two failure modes; the failure of the nut and the steel failure of the rod itself. Figure 10 shows the results gathered in the database. Figured are results over all anchor sizes (M6 to M20). It can be seen, that both failure modes occur in all failure times with a tendency to failure of the nut for high steel strength (=small failure time) and a steel failure for small steel strength (=high failure time). All results follow the same tendency, so that an evaluation of both failure modes together is reasonable.

In Figure 11, all test results from the database with threaded rods M12 are shown. An evaluation according to TR020 is done and compared to the values from simplified design model. It can be shown that the test results are much higher than the values given in TR020.
4 Conclusion

Currently the guidelines for the evaluation and execution of fire tests on bonded anchors are not clear. Furthermore, no regulations for the evaluation of chemical anchor systems with flexible anchorage depth exist. The research work conducted at the Technical University of Kaiserslautern could be a basis for changes. Besides the evaluation, the test execution has to be determined thereby.
The use of fire tests according to EAD 330087-00-0601\(^4\) in combination with thermal simulations could be an option to evaluate anchor systems with flexible anchorage depth.

Open issues on this research are the influence of cracks on bonded anchors in case of fire.

References:

1. ETAG 001 - Guideline for European Technical Approval of Metal Anchors for Use in Concrete, Part 5: Bonded Anchors, EOTA, 2013


4. EAD 330087-00-0601: Systems for post-installed rebar connections with mortar, EOTA, Draft 2015


RECENT DEVELOPMENTS IN DESIGN OF POST-INSTALLED REBAR CONNECTIONS UNDER TEMPERATURE

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ABSTRACT

A typical application for post-installed rebar connections consists in overlapping joints with existing reinforcement or anchoring of the reinforcement at a slab or beam support.

Tests at cold state show that a post-installed rebar system can develop the same bond resistance with the same safety margin as cast-in-place rebar. Consequently, anchorage length and lap length for post-installed rebars can be calculated as for cast-in-place according to the Eurocode provisions.

It is noticed that exposure to elevated temperatures affects bond properties for post-installed systems more dramatically than for cast-in-place rebars. Additionally, the decay in bond properties is product dependent.

Existing results available in literature regarding the effect of the temperature on the decay of the bond properties are reviewed and discussed. Finally, the results of an experimental campaign carried out on different types of bonding agent are presented and compared with results for cast-in samples for the same geometry and boundary conditions.

1 Introduction

In existing buildings, retrofit of existing structures or the addition of structural or non-structural elements is becoming more and more popular for both logistics and economic reasons. In reinforced concrete structures, a key role was taken by post-installed rebar connections, whose use is constantly increasing in the last decades due to their good flexibility in use and to the constantly increasing mechanical performances of the adhesives used in creating the connection.

Their main drawback consists in the possibility of designing straight anchorages only, having no chance of providing bends, hooks or welding to transverse reinforcement.

Through the years, specific design provisions have been established in order to reach the same level of safety between pre-cast and post-installed rebars¹,².

Previous investigations showed that adhesives such as chemical resins tend to have larger bond strength than traditional cast-in rebars³ and that design can be carried out according to existing code provisions for reinforced concrete⁴, due to experimental tests that show post-installed systems provides comparable bond stiffness and not lower strength than the cast-in-place rebars.
However, few studies have been carried out to establish the behavior of such connections under elevated temperatures\textsuperscript{5,6}. The significant decrease in the bond capacity detected at high temperatures\textsuperscript{5} then requires some review and discussions for the embedment depths for post-installed products.

2 Cast in place and post-installed rebar connections

Rebar connections are designed to create structural joints in a reinforced concrete (RC) structure. Key application of rebar connections are the following\textsuperscript{2}:

- An overlapping joint with existing reinforcement in a building component (Figures 1 and 2)
- Anchoring of the reinforcement at a slab or beam support; end support/bearing of a slab designed as simply supported as well as its reinforcement for restraint forces (Figure 3)
- Anchoring of reinforcement of building components stressed primarily in compression (Figure 4)
- Anchoring of reinforcement to cover the line of acting tensile force (Figure 5)

![Figure 1: Overlap joint for rebar connections of slabs and beams\textsuperscript{2}](image1)

![Figure 2: Overlap joint at a foundation of a column or wall where the rebars are stressed in tension\textsuperscript{2}](image2)

![Figure 3: End anchoring of slabs or beams, designed as simply supported\textsuperscript{2}](image3)

![Figure 4: Rebar connection for components stressed primarily in compression. The rebars are stressed in compression\textsuperscript{2}](image4)
Design may be carried out according to the same procedure adopted for cast-in rebars in Eurocode 2 (EC2), assuming that qualification tests show that post-installed rebar system can develop the same values of bond resistance with the same safety margin as cast-in-place rebars. Table 1 reports the minimum bond resistance post-installed rebar are required to achieve to design the connection with the same values of bond resistance provided by EC2.

Table 1: Required bond resistance for post-installed rebars

<table>
<thead>
<tr>
<th>Concrete strength class</th>
<th>Design values of the ultimate bond resistance according to EC2 for good bond conditions $f_{bd}$ (N/mm$^2$)</th>
<th>Required bond resistance for post-installed rebars $f_{bm\ req}$ (N/mm$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C12/15</td>
<td>1.6</td>
<td>7.1</td>
</tr>
<tr>
<td>C16/20</td>
<td>2.0</td>
<td>8.6</td>
</tr>
<tr>
<td>C20/25</td>
<td>2.3</td>
<td>10.0</td>
</tr>
<tr>
<td>C25/30</td>
<td>2.7</td>
<td>11.6</td>
</tr>
<tr>
<td>C30/37</td>
<td>3.0</td>
<td>13.1</td>
</tr>
<tr>
<td>C35/35</td>
<td>3.4</td>
<td>14.5</td>
</tr>
<tr>
<td>C40/50</td>
<td>3.7</td>
<td>15.9</td>
</tr>
<tr>
<td>C45/55</td>
<td>4.0</td>
<td>17.2</td>
</tr>
<tr>
<td>C50/60</td>
<td>4.3</td>
<td>18.4</td>
</tr>
</tbody>
</table>

As known, the value $f_{bd}$ is determined as it follows:

$$f_{bd} = 2.25 \times \eta_1 \times \eta_2 \times f_{cld}$$  \hspace{1cm} (1)

where $\eta_1$, $\eta_2$ are parameters related to bond conditions and diameter of rebar and $f_{cld}$ is the design tensile strength of concrete.

When such requirement on the bond resistance is not fulfilled, than the following limitation for using the design values of the ultimate bond stress shall be considered:
f_{bd} = f_{bm} * 0,345 \tag{2}

The reason for the difference between the values of $f_{bm}^{\text{req}}$ and $f_{bd}$ is due to that the latter is valid for “worst case conditions”, i.e. minimum concrete cover, minimum spacing, minimum amount of transverse reinforcement and large bonded length (as typical in real applications), while tests on post-installed rebars are carried out on single rebar with large concrete cover on a bonded length of ten diameters.

Therefore, when the behavior of post-installed rebars is investigated under specific conditions, a direct comparison between testing and design value is not appropriate, while it is preferable to investigate the variation with respect to a specific reference value.

This paper investigates the decay in the bond properties of both cast-in and post-installed rebar for specimens of the same geometry when subjected to the same experimental procedure.

For cast-in rebars it can be assumed that the same relationship between bond strength and tensile strength of concrete is valid for elevated or high temperatures (see Eq. 1). Consequently, the reduction factor of $f_{ck}$ provided by EC2\textsuperscript{8} (see Figure 6) can be expected to be applied to bond strength as well.

The same approach can be followed evaluating the decay of concrete tensile strength by the provisions of fib Model Code 2010\textsuperscript{7}.

![Figure 6: Reduction factor $k_{ct}$ for concrete characteristic tensile strength as a function of temperature](image)

### 3 Experimental investigations

Recently, a testing and assessment procedure was developed by EOTA (European Organization for Technical Assessment) to qualify post-installed rebars under temperature in a relevant EAD (European Assessment Document) – Systems for post-installed rebar connections with mortar\textsuperscript{9} (which additionally supersedes TR023\textsuperscript{2} for the behavior at cold state). The test procedure presented in such EAD was adopted in the present study for both post-installed and cast-in rebars, by introducing some specific adjustments for the latter case, as it will be described in the following paragraphs.
3.1 Test materials
Concrete of class C20/25 was produced according to the specifications of EAD 330087\textsuperscript{8}. Mix components of concrete are reported in Table 2. The steel elements of the connections are $\Phi 12$ mm ribbed bars made of B450C carbon steel.

Table 2: Mix components

<table>
<thead>
<tr>
<th>Component</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement CEM I 52,5 R (kg/m\textsuperscript{3})</td>
<td>250</td>
</tr>
<tr>
<td>Aggregate (kg/m\textsuperscript{3})</td>
<td></td>
</tr>
<tr>
<td>Fine (0-4)</td>
<td>300</td>
</tr>
<tr>
<td>Fine (0-8)</td>
<td>700</td>
</tr>
<tr>
<td>Coarse (8-15)</td>
<td>270</td>
</tr>
<tr>
<td>Coarse (15-25)</td>
<td>600</td>
</tr>
<tr>
<td>Total aggregate (kg/m\textsuperscript{3})</td>
<td>1870</td>
</tr>
<tr>
<td>Superplasticizer (% on cement mass)</td>
<td>0</td>
</tr>
<tr>
<td>Water (l/m\textsuperscript{3})</td>
<td>190</td>
</tr>
<tr>
<td>Water/cement</td>
<td>0.76</td>
</tr>
</tbody>
</table>

3.2 Test procedures
The tests were performed on steel encased C20/25 non-cracked concrete cylinders with diameter of 150 mm and height of 200 mm. Two points for temperature measurement (thermocouples type K) were placed at 120 mm from the top of the hole and at 10 mm below the concrete surface (Figure 7). The rebars were installed in the axis of the concrete cylinders following the manufacturer installation instructions. The rebars were protected outside of the cylinder to avoid steel failure. This test setup was used for both pre-cast (Figure 8) and post-installed (Figure 9 and 10) rebar connections.

Figure 7: Details of test specimen (acc. to EOTA EAD 330087, 2015)
An electrical oven able to reach a maximum temperature of 850 °C was used (Figure 11). The specimen was set inside the oven, then a series of four additional layers (insulating material + steel plate + insulating material + steel plate) were interposed between the bottom face of the specimen and the bottom side of the chamber to both assure a proper insulation and an even distribution of the contact stresses. The chamber was successively closed by means of additional insulating material.

The rebar was loaded up by an external actuator with an increasing centric axial force until the value of target load. After reaching the target load the temperature in the oven was increased at a rate of approximately 10 °C/min until failure occurred. The total test duration then varies according to the value of the applied load.

3.3 Experimental results and discussion

Table 3 and Table 4 report experimental results for pre-cast and post-installed rebar, respectively.

Both tables report the sample reference code, the applied load \( N_{sust} \) or the peak load \( N_{max} \) (for reference tests at room temperature only), the temperature at failure \( T_{failure} \), the maximum difference between the two thermocouples during the tests \( \Delta T_{max} \), the difference between the two thermocouples at failure \( \Delta T_{test} \), the average bond strength \( \tau_m \), the total test duration. The average bond strength reported is calculated according to the following equation:
\[ \tau_m = \frac{N_m}{(\pi \cdot d \cdot l_v)} \]  \hspace{1cm} (3)

with \( N_m \) assuming the value of \( N_{\text{sust}} \) or \( N_{\text{max}} \), depending on type of test, \( l_v \) the embedment depth and \( d \) the diameter of rebar. For post-installed rebars 4 type of resins were used, Product 1 (PI1) and Product 3 (PI3) are vinylester types while Product 2 (PI2) and Product 4 (PI4) are epoxy types.

### Table 3: Test results on pre-cast rebars

<table>
<thead>
<tr>
<th>Code</th>
<th>Test procedure</th>
<th>( N_{\text{sust}} / N_{\text{max}} )</th>
<th>( T_{\text{failure}} )</th>
<th>( \Delta T_{\text{max}} )</th>
<th>( \Delta T_{\text{test}} )</th>
<th>( \tau )</th>
<th>Test duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC_D12_CL_01</td>
<td>RT</td>
<td>36.0, 20.0</td>
<td>-</td>
<td>-</td>
<td>7.96</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>PC_D12_CL_02</td>
<td>RT</td>
<td>35.1, 20.0</td>
<td>-</td>
<td>-</td>
<td>7.77</td>
<td>-</td>
<td>3</td>
</tr>
<tr>
<td>PC_D12_CL_03</td>
<td>CL</td>
<td>22.6, 78.8</td>
<td>9.12</td>
<td>8.41</td>
<td>5.00</td>
<td>0</td>
<td>50</td>
</tr>
<tr>
<td>PC_D12_CL_04</td>
<td>CL</td>
<td>18.1, 247.2</td>
<td>33.66</td>
<td>33.65</td>
<td>4.00</td>
<td>1</td>
<td>49</td>
</tr>
<tr>
<td>PC_D12_CL_05</td>
<td>CL</td>
<td>24.9, 97.4</td>
<td>29.62</td>
<td>28.85</td>
<td>5.50</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>PC_D12_CL_06</td>
<td>CL</td>
<td>20.4, 135.1</td>
<td>19.88</td>
<td>6.40</td>
<td>4.50</td>
<td>1</td>
<td>32</td>
</tr>
<tr>
<td>PC_D12_CL_07</td>
<td>CL</td>
<td>27.1, 76.6</td>
<td>8.87</td>
<td>6.37</td>
<td>6.00</td>
<td>0</td>
<td>53</td>
</tr>
</tbody>
</table>

### Table 4: Test results on post-installed rebars

<table>
<thead>
<tr>
<th>Code</th>
<th>Test procedure</th>
<th>( N_{\text{sust}} / N_{\text{max}} )</th>
<th>( T_{\text{failure}} )</th>
<th>( \Delta T_{\text{max}} )</th>
<th>( \Delta T_{\text{test}} )</th>
<th>( \tau )</th>
<th>Test duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI1_D12_CL_01</td>
<td>CL</td>
<td>58.8, 29.7</td>
<td>3.07</td>
<td>2.07</td>
<td>13.00</td>
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<td>14</td>
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<tr>
<td>PI1_D12_CL_02</td>
<td>CL</td>
<td>39.3, 62.8</td>
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<td>4.95</td>
<td>8.69</td>
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<td>CL</td>
<td>30.0, 70.0</td>
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<td>PI1_D12_CL_04</td>
<td>CL</td>
<td>13.6, 150.0</td>
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<td>45</td>
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<tr>
<td>PI2_D12_CL_01</td>
<td>CL</td>
<td>53.4, 36.3</td>
<td>8.88</td>
<td>7.39</td>
<td>11.81</td>
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<td>13</td>
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<td>CL</td>
<td>34.4, 46.2</td>
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<td>7.61</td>
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<td>PI2_D12_CL_03</td>
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<td>20.0, 53.6</td>
<td>5.27</td>
<td>5.27</td>
<td>4.42</td>
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<td>PI2_D12_CL_04</td>
<td>CL</td>
<td>14.0, 60.5</td>
<td>7.02</td>
<td>4.57</td>
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<td>PI3_D12_CL_01</td>
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<td>18.0, 144.8</td>
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<td>17.76</td>
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<td>27.0, 115.5</td>
<td>17.05</td>
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<td>5.97</td>
<td>-</td>
<td>58</td>
</tr>
<tr>
<td>PI3_D12_CL_03</td>
<td>CL</td>
<td>54.0, 47.0</td>
<td>3.57</td>
<td>0.48</td>
<td>11.94</td>
<td>-</td>
<td>32</td>
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<tr>
<td>PI3_D12_CL_04</td>
<td>CL</td>
<td>13.5, 176.4</td>
<td>18.78</td>
<td>18.78</td>
<td>2.99</td>
<td>1</td>
<td>39</td>
</tr>
<tr>
<td>PI3_D12_CL_05</td>
<td>CL</td>
<td>22.5, 128.9</td>
<td>11.28</td>
<td>2.45</td>
<td>4.98</td>
<td>1</td>
<td>11</td>
</tr>
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<td>PI3_D12_CL_06</td>
<td>CL</td>
<td>36.0, 79.8</td>
<td>4.88</td>
<td>2.88</td>
<td>7.96</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
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<td>45.0, 49.3</td>
<td>4.05</td>
<td>2.92</td>
<td>9.95</td>
<td>-</td>
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<tr>
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<td>CL</td>
<td>18.0, 59.7</td>
<td>7.03</td>
<td>5.44</td>
<td>3.98</td>
<td>-</td>
<td>56</td>
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<tr>
<td>PI4_D12_CL_02</td>
<td>CL</td>
<td>27.0, 59.1</td>
<td>1.46</td>
<td>1.16</td>
<td>5.97</td>
<td>-</td>
<td>53</td>
</tr>
<tr>
<td>PI4_D12_CL_03</td>
<td>CL</td>
<td>46.0, 47.8</td>
<td>1.07</td>
<td>0.97</td>
<td>10.17</td>
<td>-</td>
<td>48</td>
</tr>
<tr>
<td>PI4_D12_CL_04</td>
<td>CL</td>
<td>36.0, 52.3</td>
<td>3.51</td>
<td>3.51</td>
<td>7.96</td>
<td>-</td>
<td>48</td>
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<tr>
<td>PI4_D12_CL_05</td>
<td>CL</td>
<td>9.0, 92.4</td>
<td>5.34</td>
<td>5.25</td>
<td>1.99</td>
<td>1</td>
<td>5</td>
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<tr>
<td>PI4_D12_CL_06</td>
<td>CL</td>
<td>40.7, 51.9</td>
<td>2.54</td>
<td>2.54</td>
<td>9.00</td>
<td>-</td>
<td>46</td>
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<td>PI4_D12_CL_07</td>
<td>CL</td>
<td>31.7, 57.4</td>
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<td>3.50</td>
<td>7.00</td>
<td>-</td>
<td>56</td>
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<td>PI4_D12_CL_08</td>
<td>CL</td>
<td>22.6, 61.1</td>
<td>9.10</td>
<td>1.45</td>
<td>5.00</td>
<td>-</td>
<td>34</td>
</tr>
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<td>PI4_D12_CL_09</td>
<td>CL</td>
<td>13.6, 66.5</td>
<td>16.00</td>
<td>15.79</td>
<td>3.00</td>
<td>-</td>
<td>35</td>
</tr>
</tbody>
</table>
The reported temperature values are calculated as the mean between the values measured by the thermocouples along the bar. Figure 12-13 show the variation in temperature at the thermocouples location during heat treatment for two pre-cast and post-installed samples, respectively. It can be noticed how the distribution of temperature along the bar is quite uniform.

Figure 12: Air and rebar temperatures during cast-in testing

Figure 13: Air and rebar temperatures during post-installed testing

Figure 14 reports test results in terms of bond strength vs temperature curves. For temperature range between 20°C and 50 °C it is possible to note how post-installed rebars show a higher bond strength than cast-in samples. This trend changes when increasing temperature; post-installed rebars show a rapid decay in bond strength also for relatively low values of temperature. This decay is strongly product dependent, even if it can be noticed that vinylester products show better performances than epoxy resins.

A reduction factor of the bond strength as a function of temperatures \( k_f(T) \) can be calculated as

\[
k_f(T) = \frac{\tau(T)}{\tau(21 \, ^\circ C)} \leq 1.0 \quad \text{for} \ 21 ^\circ C \leq T \leq T_{\text{max}} \tag{4}
\]

\[
k_f(T) = 0 \quad \text{for} \ T > T_{\text{max}} \tag{5}
\]

Figure 15 report the reduction factor \( k_f(T) \) evaluated for all the tests. Results are compared with trend derived by EC2\(^8\) and Model Code 2010\(^7\).
It is noticed how the decay in bond strength for post-installed rebar cannot be predicted by any available code. Additionally, being strongly product-dependent, specific testing is required for any different adhesive. The reduction factor evaluated for cast-in rebar is significantly overestimated by Eurocode 2, while Model Code 2010 seems to be on the safe side.

Finally, results for cast-in rebars are additionally compared with results available in literature (Figure 16). Test results from Ergun et al. and Lubloy and Hlavicka basically show a slight reduction in bond strength up to 200 °C and a decay which is well predicted by the models of Aslani and Samali, while Huang model describes trend until 600°C.

In all the cases the decay in bond strength is significantly lower than the one presented in this study. However, it must be noticed that in such studies only residual test after exposure at high temperatures were accounted, while no test was carried out with a constant load applied. Consequently, the effect of creep it totally neglected, which seems to play a significant role in this investigation. On the other hand, the adopted bond length were lower than the ones used in study and they allowed a more uniform distribution of stress along the bar, which could make the results coming from the two approaches not comparable.

In the future, the test setup presented in this paper may be adapted to account also for lower values of bonded length, while still allowing the application of a constant load during the temperature increase.

Figure 16: Reduction factor $k_f(T)$ - Comparison with literature results

4 Conclusions

In this paper an experimental investigation on the decay of bond properties of both pre-cast and post-installed rebars under temperature is presented. The comparison between pre-cast and post-installed rebars suggests that the behavior at lower temperatures of post-installed is comparable with pre-cast rebar, while when increasing temperature some adhesives show a decay in bond strength more significant than pre-cast, hence the behavior of post-installed is strongly product dependent. Generally, epoxy resins show a stronger decay than vinylester adhesives.

The comparison between tested and predicted values for cast-in rebars suggest that Model Code 2010 better fits the evaluated reduction factor than Eurocode 2, which seems to overestimate behavior of
rebar connection when increasing temperature, so as test results available in literature. However, different test procedures, samples dimensions and boundary and loading conditions are compared; it is consequently suggested to adapt the presented test setup to account also for lower values of bonded length.

5 Acknowledgements

The authors gratefully thank all the members of the technical staff of LPMSC, sector Structural Anchors. Special thanks to Eng. Andrea Rocca for his contribution to the test campaign.

References:


9. EOTA EAD 330087-00-06.01,“Systems for post-installed rebar connections with mortar”, European Organization for Technical Assessment (EOTA), Brussels, Belgium, 2015


CONCRETE EDGE FAILURE OF FASTENERS AFTER FIRE EXPOSURE EXPERIMENTAL AND 3D FE STUDY

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ABSTRACT

In the present study different single headed stud anchors are exposed to various duration of fire and loaded in shear against the free concrete edge up to failure. The concrete edge distances of 50 mm, 100 mm and 150 mm are considered. The standard fire curve ISO 834 with the corresponding duration time of 15, 30, 60 and 90 min were employed to investigate the effect of fire on the reduction of load bearing capacity of fasteners. The load bearing capacity after 90 min of fire exposure and reference values without fire exposure were obtained experimentally. The experimental results were numerically simulated and verified using 3D finite element code based on the thermo-mechanical model for concrete. The model is based on the microplane constitutive law and the behaviour of steel is assumed to be linear elastic. After verification of the model, the extensive numerical parametric study is carried out. It is shown that the used 3D finite element code can realistically replicate the residual capacity of fasteners that were damaged through the fire exposure of concrete member. The fire exposure leads to significant reduction of their shear failure capacity. After only 15 min of fire duration, the reduction can be more than 50 % of the reference value. The results indicate that the highest reduction of capacity is obtained for the cold state, i.e. when the concrete is after fire cooled down to the room temperature. The minimum resistance is reached already after 60 min of fire and the relative reduction of capacity is higher for larger edge distances.

1 Introduction

It is known that the strength of concrete is substantially reduced with fire exposure. This leads to strong influence on the capacity of connections between steel and concrete, namely, the load-bearing behaviour of fasteners in concrete considering fire safety. Particular cases are that when the fasteners are located close to the edge of a concrete member and loaded in shear towards free edge. After fire exposure the concrete becomes damaged, which can result in strong reduction of failure capacity of fasteners. Currently in the literature there are only a few experimental and numerical studies devoted to the residual capacity of fasteners after fire exposure of concrete. Periškić performed experimental tests and numerical simulations on the load-bearing behaviour of single anchor and group anchors located close to concrete edge and away from concrete edge under tensile load at high temperatures, considering the fire exposure on one side and two sides of the edge. It was shown that for anchors located away from concrete edge, the embedment depth of anchor plays a main role on the residual capacity of anchors loaded in tension under fire exposure, the larger the embedment depth is, the
smaller the reduction is. When the anchor is located close to edge, the two-sided fire on the edge induces strong reduction of the tensile capacity under fire exposure, which is reasonable due to the thermal penetration of heat into concrete member from both sides of the edge. From this point of view, the concrete edge failure of anchors loaded in shear towards free edge may represent the most critical case, when considering the fire safety of fasteners near a concrete edge.

The previously conducted experimental tests confirmed that the post-fire capacity, which was obtained after cooling from 90 min of fire exposure, reduces remarkably. Therefore, in the present study the load bearing capacity of headed stud anchors loaded in shear towards the free edge after fire exposure of different durations is experimentally and numerically investigated. Firstly, the experimental results for 90 min of fire exposure and then the numerical results for different ISO 834 fire durations are presented and discussed.

2 Experimental Study

The fire tests were conducted at the fire safety laboratory (MPA), University of Stuttgart. The results for anchor configurations with different edge distances ($c = 50 \text{ mm}, 100 \text{ mm and } 150 \text{ mm}$), shaft diameters ($d = 16 \text{ mm and } 25 \text{ mm}$) and embedment depth $h_{ef} = 95 \text{ mm}$ under two-sided fire exposure are presented.

2.1 Experimental Program

![Diagram of anchor configuration](image)

Figure 1: (a) Applied fire on two sides of the concrete edge and (b) anchor loaded in shear after cooling.

Figure 1a schematically shows the applied fire exposure on the concrete edge from the top and the front sides. The fire test were carried out according to EN 1363-1\textsuperscript{5}, in which the fire curve is according to ISO 834\textsuperscript{6}. The temperature inside the furnace was controlled to rise until 90 min and then it was cooled down to room temperature naturally. Afterwards, the anchor was loaded perpendicular to the free edge in shear according to the guideline\textsuperscript{7}, with sufficient support spacing as shown in Fig. 1b. The distance between the two supports was kept at 5 times the edge distance of the anchor. The displacement of the anchor in loading direction was acquired by using linear variable displacement transducers (LVDT). To avoid concrete spalling at high temperature, low strength
concrete of C12/15 was used. The material properties are summarized in Tab. 1 and the same are used in the numerical simulations.

Table 1 Material properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s modulus (GPa)</th>
<th>Poisson’s ratio</th>
<th>Uniaxial compressive strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Fracture energy (J/m²)</th>
<th>Heat conductivity (W/mK)</th>
<th>Heat capacity (J/kgK)</th>
<th>Weight density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>27</td>
<td>0.18</td>
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<td>1.6</td>
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<td>2300</td>
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<tr>
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<td>-</td>
<td>-</td>
<td>18.00</td>
<td>500</td>
<td>7900</td>
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</tbody>
</table>

2.2 Experimental Results

Experimental results show that the shear loading capacity of headed stud anchor reduces remarkably after 90 min of fire exposure. Figure 2a shows the load-displacement curves of anchors loaded in shear at room temperature. As can be seen, the initial stiffnesses are basically the same for anchors with edge distance of 50 mm and 150 mm. However, after cooling from 90 min fire exposure, there is a strong reduction of resistance (Fig. 2b). As can be seen, the larger edge distance leads to stiffer ascending part of the load-displacement curve.

Figure 2: Load-displacement curves of anchors (a) at room temperature and (b) after 90 min of fire exposure (number 2 in the legend indicates 2-sided fire).
The typical failure pattern of concrete edge is shown in Fig. 3a. Due to the temperature induced damage of concrete, the development of the cracks is rather irregular. As shown in Fig. 3a, one crack initiates from the position of the anchor, which appeared after the loading and indicating the peak load. Another relatively large crack formed at about 100 mm away from the anchor on the back side, which is initiated from the head of the anchor embedded in the concrete. In this case the angle $\alpha$ with respect to the edge is less than 10° that is much less than 35°, which is typical for concrete at room temperature. From Fig. 3b strong reduction of peak loads after 90 min of fire exposure can be seen. It was found that the capacity, regardless of the capacity before fire exposure, reduced to less than 10 kN after 90 min of fire exposure.

3 3D Finite Element Analysis

To investigate and better understand the behaviour and failure capacity of anchors at elevated temperature that are loaded in shear against free concrete edge, a 3D numerical finite element study was carried out. In the study the edge distance was varied as $c = 50$ mm, 100 mm, and 150 mm for fire exposure of 15 min, 30 min, 60 min and 90 min. The analysis was carried out by using a 3D thermo-mechanical FE code MASA. The constitutive law for concrete is the temperature-dependent microplane model.

3.1 Numerical Model

In the numerical analysis only two-sided fire exposure was considered. As shown in Fig. 4a, the heating area covers an area that has 7 times the edge distance along the edge and two times the edge distance to the back side of the anchor. According to Eligehausen et al. this area is sufficient to cover the fracturing area due to loading in shear of anchor towards free edge. The supports and main constraints are shown in Fig. 4a. It should be noted that for the analysis of the cold state the steel plate was set as extremely low stiffness value (E-modulus: 1 GPa) during the heating and cooling phase and it was set to the normal value (E-modulus: 200 GPa) of steel while loading up to failure in cold state. The reason is due to the fact that during heating and cooling the anchor plate was not installed. While for hot state analysis, the stiffness value of steel plate was kept as normal value of
steel (E-modulus: 200 GPa), due to the fact that the steel plate was always installed for hot state test or in reality.

Figure 4b shows that the numerical results agree very well with the experimental results, this applies to condition at room temperature, 90 min of fire (hot state) and after fire exposure (cold state). A detailed verification analyses can be found in the literature\textsuperscript{11}. Note, however, that due to the boundary conditions the reduction of resistance after 90 min of fire in hot state was not significant. For more detail see Tian et al.\textsuperscript{11}.

![Figure 4](image)

**Figure 4**: (a) Finite element model and discretization for analysing load-bearing behaviour of anchor under different fire exposure and (b) validation of the numerical model.

### 3.2 Numerical Results

The numerically obtained load-displacement curves of anchors loaded in shear at room temperature (reference), in hot state immediately after fire exposure and in cold state after cooling from fire exposure are plotted in Fig. 5. They show that a large loss of stiffness is observed due to fire exposure. The loss of stiffness increases with increasing fire duration, both in hot state and cold state. If one compares the hot state and cold state behaviour for the same anchor, it is obvious that the load-bearing behaviour in hot state is much stiffer than that in cold state. But this difference seems to diminish with increasing edge distance. The loss of stiffness of anchor loaded in shear after fire exposure can be attributed to the known behaviour of concrete material at elevated temperatures\textsuperscript{1,12}. Meanwhile, the larger loss of stiffness in cold state compared to that in hot state reflects that concrete material possesses higher stiffness in hot state than that in cold state, which can also be seen from the results tested by Klingsch et al.\textsuperscript{13}.
Figure 5: Load-displacement curves obtained from the numerical analyses of anchors loaded in shear after fire exposure: (a) c50h95d25 in hot state and (b) c50h95d25 in cold state; (c) c100h95d25 in hot state and (d) c100h95d25 in cold state; (e) c150h95d25 in hot state (f) c150h95d25 in cold state.
From Fig. 5 can be seen that the peak loads are strongly reduced with fire duration. The peak loads from both numerical and experimental analyses are plotted against fire duration $f_i(t)$ in Fig. 6a. It is shown that the reduction of peak loads after fire exposure is very large already after 15 min of fire exposure. The capacity for each anchor in hot state is slightly higher than that in cold state. It can be seen that a relatively large difference exists in the reduction of peak loads for anchors with different edge distances. However, this relative large difference in the peak toad at room temperature after fire of 90 min completely disappear, i.e. after 90 min of fire load the resistance is strongly reduced and it becomes almost independent on the edge distance. The experimental results presented in Fig. 6a also proved this tendency.

The relative values in terms of those at room temperature are plotted as a function of fire duration $f_i(t)$ in Fig. 6b. In case of small edge distance $c = 50$ mm, the reduction in terms of reference value at room temperature is about 44%, 54%, 66% and 79% after 15 min, 30 min, 60 min and 90 min of fire exposure, respectively. For anchors with relatively large edge distance $c = 150$ mm, the reduction is about 61%, 68%, 85% and 90% after 15 min, 30 min, 60 min 90 min of fire exposure, respectively. The results show that relative resistance of failure capacity increases with edge distance and that after 90 min of fire exposure the resistance reaches the minimum. Note that in the current Eurocode 2 Part 4⁴ only 90 min and 120 min are considered for the design of fasteners under fire. However, according to the presented results it would be more appropriate to find the reduction law for concrete edge resistance within 90 min of fire duration. It is obvious that the data uniformly show quite good nonlinear reduction. Based on the numerical data (including those from both hot state and cold state analyses) a regression analysis was performed and the following equation is obtained for the reduction of resistance (Fig. 6b):

for $c = 50$ mm: $V_{u,c,f_i(t)}/V_{u,c,f_i(0)} = 0.25 + 0.75 \times 0.95^t$  \hspace{1cm} (1)

for $c = 150$ mm: $V_{u,c,f_i(t)}/V_{u,c,f_i(0)} = 0.10 + 0.90 \times 0.93^t$  \hspace{1cm} (2)
The numerical results should be further verified by experimental results obtained from near edge installed anchors loaded in shear after different fire duration.

4 Conclusion

In the present study the results of experimental and numerical investigations on a single headed stud anchors close to concrete edge exposed to different duration of fire and then loaded perpendicular to free edge in shear are presented and discussed. Numerical analyses were performed at room temperature, after 15 min, 30 min, 60 min and 90 min of fire duration (hot and cold state). In combination with experimental results after 90 min of fire duration and reference values without fire exposure, the following can be concluded. (1) The employed numerical model is able to realistically replicate the experimental results; (2) The edge distance is the main factor that influences the capacity of anchor loaded in shear towards free edge. Compared to the failure at room temperature, the crack development at elevated temperature is more irregular due to the temperature induced damage of concrete; (3) Shear loaded anchors largely lose their stiffness after fire exposure. The longer the fire duration is, the larger is the loss. In hot state the anchor behaves much stiffer than in the cold state, however, this difference diminishes with increasing edge distance; (4) The analysis shows that the shear resistance of anchors in the hot state is slightly higher than the resistance in the cold state under each fire duration; (5) The peak load is reduced significantly already after only 15 min of fire exposure. The relative reduction of capacity increases with increase of edge distance. After 90 min of fire exposure the analysis shows that the capacity of anchors with different edge distances reaches minimum resistance; (6) Further experimental and numerical studies are needed to confirm the presented results.

5 Acknowledgement

The authors would like to thank the German Research Foundation (DFG) and the China Scholarship Council for providing funding for this project.

References:


CHEMICALLY-BONDED POST-INSTALLED STEEL REBARS IN A FULL-SCALE SLAB-WALL CONNECTION SUBJECTED TO THE STANDARD FIRE (ISO 834-1)

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ABSTRACT

The improvement in mechanical and adhesion properties of polymer resins allowed to progressively substitute cast-in place rebars by chemically-bonded post-installed rebars in some applications, by providing equivalent or even higher mechanical properties at ambient temperature. However, the mechanical behavior of post-installed rebars is mainly governed by the mechanical behavior of polymer resins, which are highly sensitive to temperature, addressing the topic of fire safety. In 2015, the European Assessment Document 330087-00-06.01 has been issued to harmonize the evaluation method on fire performance of these polymeric adhesives. This evaluation document proposes a fire design method allowing to ensure a sufficient bond resistance for a given duration of fire exposure before collapse.

This paper presents a full-scale fire test performed in the “Vuclain” modular gas furnace at CSTB Champs-sur-Marne (France), carried out on a 2.94 m x 2 m x 0.15 m cantilever concrete slab connected to a wall by chemically-bonded post-installed rebars and exposed to ISO fire 834-1 until failure. Experimental investigations showed that the design method presented in this paper is able to predict a time of slab collapse very close to that obtained by the fire test. The interpretation of the data collected during the fire test revealed the presence of thermal and physical phenomena responsible for the decay in the fire resistance of the post-installed rebars and therefore, responsible for the slab collapse under fire exposure.

1 Introduction

Post-installed rebars (PIRs) is a structural joining technique allowing the connection and the load transfer between structural elements using steel rebars and adhesive polymers1. PIRs were initially used in concrete constructions in retrofitting, extension and in repairing structures by adding new concrete sections to existing elements2,3. Over the time, the improvement in mechanical and adhesion properties of polymer adhesives have allowed to enhance the mechanical behavior of PIRs and led to achieve equivalent or even higher mechanical responses than cast-in place rebars at normal operating temperatures4,5. Thus, PIRs have gradually substitute cast-in place rebars in new constructions for
some applications by offering advantageous solutions and flexibility allowing to meet the high architectural requirements. However, the mechanical behavior of post-installed rebars is directly linked to the mechanical properties of polymer resins, which are highly sensitive to temperature. Consequently, the temperature increase at the PIRs presents a potential risk affecting their safety use. Therefore, fire presents a serious hazard which should be considered when designing PIRs. Very few regulations and technical documents exist today proposing methods to assess and to design the fire resistance of chemically-bonded post-installed rebars. The European Assessment Document 330087-00-06.01 is one of the most important documents existing today, proposing a fire design method for PIRs in fire situation.

This paper presents a full-scale fire test performed on “Vulcain” modular gas furnace of CSTB, carried out on a 2.94 m x 2 m x 0.15 m cantilever concrete slab connected to a wall by 8 PIRs and exposed to ISO fire 834-1 until its failure (Figure 1). The main goal is to validate a suggested design method allowing to predict the fire resistance of PIRs using characterization tests. The first part of this paper describes the test configuration and explains the design method. While, the second part presents the experimental results obtained during the fire test and compares between predicted and measured time of collapse.

Figure 1: Vulcain fire test configuration: Instrumentation and set up on the furnace

2 Test specimen conception and fire resistance prediction

2.1 Test specimen conception

A full-scale ISO fire 834-1 test was carried out at CSTB Champs-sur-Marne on the “Vulcain” modular gas furnace. This furnace offers three possible exposure areas to perform fire tests on a horizontal configuration: 3m x 3m, 4m x 3m and 7m x 3m. The chosen configuration was 4m x 3m. The set-up of the test specimen on the furnace required the presence of a concrete frame allowing to close the furnace and to ensure its fire integrity during the test. The concrete frame was made from a C45/55 reinforced concrete and represents the wall in which post-installed rebars were anchored. The
test specimen was composed of a 2.94 m x 2 m x 0.15 m cantilever slab made from C20/25 reinforced concrete containing propylene fibers in order to avoid spalling, connected to the wall by 8 post-installed rebars chemically-bonded into concrete using epoxy resin (Figure 1). Characterization tests were carried out on cubic concrete samples (150 mm x 150 mm x 150 mm) after 28 days of curing under ambient temperature and moisture conditions. The concrete used for the frame had a compressive strength of 58.7 MPa (± 1.3 MPa) and a density equal to 2263 kg/m³ (± 7 kg/m³), while the concrete used for the cantilever slab had a compressive strength of 22.4 MPa (± 0.3 MPa) and a density equal to 1987 kg/m³ (± 5 kg/m³). The number of post-installed rebars was determined according to the EC2 part 1-1 design rules¹⁰ which allow a maximum spacing between bonded rebars equal to two times the thickness of the slab. The diameter of the steel rebars was 16 mm. The embedment length was set at 135 mm in order to ensure a failure by rebars sliding during the fire test. The rebars were positioned at 100 mm height in the cantilever slab, respecting the minimum concrete cover authorized by the EC2¹⁰.

A spacing of 80 mm at the sides and 200 mm at the free end of the cantilever slab was left to take into account the concrete thermal expansion and to prevent the blocking of the cantilever slab against the frame during fire test (Figure 2). This spacing between the slab and the concrete frame was filled with thermal insulation material in order to confine heat during the fire test. The slab was mechanically loaded with 325 kg dead weight positioned at 2200 mm from the wall (Figure 1). The slab weight was estimated equal to 1753 kg. The slab weight and the dead load generated a total bending moment at the wall/slab interface equal to Mₜₒₜ = 32.28 kNm. The fire test was performed after 3 months of the concrete casting.

2.2 Prediction of the fire resistance duration

A fire design method is presented in this paper allowing to predict the fire resistance of post-installed rebars. This method relies on the knowledge and the determination of several parameters. The fire resistance calculation of PIRs using the suggested method is composed of 5 steps.
2.2.1 Calculation of the applied tensile load $F_{app}$

The determination of the tensile load applied on each rebar can be done either by analytical calculations or by finite element analysis. This step requires the knowledge of certain parameters such as the geometrical parameters of the slab, the concrete density, the applied mechanical load, the load position and the rebars position in the slab… The determination of the applied tensile load is based on the assumption of a uniform load distribution between the rebars. Calculations showed that for the studied configuration, the tensile load applied on each rebar is around $48 \text{ kN (± 3 kN)}$. 

2.2.2 Thermal calculations $\theta(x,t)$

This step consists in determining the temperature distribution in the test specimen at different moments of fire exposure using thermal calculations. Thermal calculations can be done either by finite element analysis, or by analytical calculations using finite difference method by solving Fourier’s equation (1). These two calculation methods require input data describing the variation of materials thermal properties ($\lambda(\theta)$, $C_p(\theta)$ and $\rho(\theta)$), which can be obtained directly from the Eurocode$^{10}$.

\[
\rho(\theta(x,t))C_p(\theta(x,t)) \frac{\partial \theta(x,t)}{\partial t} = \lambda(\theta(x,t)) \frac{\partial^2 \theta(x,t)}{\partial x^2} + h(\theta_{ext}(t) - \theta_{sur}(t)) + \sigma \varepsilon (\theta_{ext}^4(t) - \theta_{sur}^4(t))
\]

Where $\rho$ is the material density [kg/m$^3$]

$C_p$ is the material specific heat [J. K$^{-1}$. Kg$^{-1}$]

$\lambda$ is the material conductivity [W. m$^{-1}$. K$^{-1}$]

$\theta(x,t)$ is the temperature of an element of the PIR at position $x$ and at time $t$ [K]

$h$ is the heat transfer coefficient [W.m$^{-2}$.K$^{-1}$]

$\sigma$ is the Stefan-Boltzmann constant [W.m$^{-2}$.K$^{-4}$]

$\varepsilon$ is the emissivity of the material

$\theta_{sur}$ is the temperature at the surface of the material [K]

At the end of this first step, a temperature map is obtained, indicating the exact temperature values at every point of the test specimen and at different moments of fire exposure. Figure 3 represents the evolution of the temperature along the embedded part of the steel rebar for different moments of fire exposure.

![Figure 3: Evolution of the temperature along the embedded part of the steel rebar during fire test](image-url)
2.2.3 Bond resistance-Temperature relationship $\tau_{\text{max}}(\theta)$

The bond resistance-temperature relationship is obtained by performing pull-out tests at different temperatures on post-installed rebars anchored in concrete cylinders. These tests consist in applying a constant load on the steel rebar and then heating the test specimen progressively until the extraction of the rebar (Figure 4). This test procedure, called “pull-out tests at constant load”, provides a failure temperature for selected amounts of shear stress applied on the adhesive joint. Results obtained by pull-out tests at constant load are shown in Figure 5.

![Figure 4: Test procedure of pull-out test at constant load](image1)

![Figure 5: Results of the pull-out test campaign](image2)

2.2.4 Determination of the PIR bearing capacity $F_t$

The calculation of the evolution of the PIR bearing capacity during fire exposure is carried out in two stages. The first stage consists in associating a bond resistance value to each element of the bonded rebar at different moments of fire exposure by knowing the temperature evolution obtained by thermal calculations and by using the relationship bond resistance-PIR temperature (Figure 5). The second step consists in summing the bond resistance values along the embedded part of the rebar for a given moment of the fire exposure, as described by equation (2).

$$F_t = 2\pi r \int_0^L \tau_{\text{max}}(\theta(x,t)) \, dx$$

Where $F_t$ is the load bearing capacity of the PIR at time $t$ [N]
- $r$ is the radius of the steel rebar [mm]
- $L$ is the embedment length [mm]
- $\tau_{\text{max}}$ is the bond resistance obtained by pull-out tests [MPa]
- $\theta(x,t)$ is the temperature of an element of the PIR at position $x$ and at time $t$ [K].

Table 1 summarizes the evolution of the calculated bearing capacity for every 30 minutes of fire exposure.
Table 1: Evolution of the load bearing capacity of the PIR during the fire test

<table>
<thead>
<tr>
<th>Fire exposure [min]</th>
<th>Load bearing capacity [kN]</th>
<th>Applied load [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>190</td>
<td>&gt; 48</td>
</tr>
<tr>
<td>30</td>
<td>190</td>
<td>&gt; 48</td>
</tr>
<tr>
<td>60</td>
<td>131</td>
<td>&gt; 48</td>
</tr>
<tr>
<td>90</td>
<td>79</td>
<td>&gt; 48</td>
</tr>
<tr>
<td>120</td>
<td>40</td>
<td>&lt; 48</td>
</tr>
<tr>
<td>180</td>
<td>9</td>
<td>&lt; 48</td>
</tr>
</tbody>
</table>

2.2.5 Time of collapse
The fire resistance design method assumes that failure occurs when the shear stress reaches the bond resistance at all the elements of the PIR. Therefore, the time collapse is considered as the time at which the PIR bearing capacity becomes equal to or lower than the applied tensile. Thus, according to calculations, the failure must occur around 120 minutes of fire exposure.

3 Validation test: Full-scale fire test on a slab-wall connection

3.1 Metrology

3.1.1 Temperature measurements
In order to study the temperature increase during the fire test, rows of thermocouples have been installed at different positions in the test specimen (Figure 1). 5 rows of thermocouples were introduced horizontally into the wall, at the same height as the rebars and positioned respectively at 111 mm, 555 mm, 1000 mm, 1450 mm and 1889 mm from the lateral side of the slab. Each row was composed of 5 thermocouples positioned respectively at 10 mm, 20 mm, 30 mm, 100 mm and 150 mm depth. These thermocouples measure the temperature increase at the PIRs during the fire test.

3 rows of thermocouples were introduced vertically at the mid-width into the slab, at 500 mm, 1500 mm and 2440 mm from the wall respectively. Each row was composed of 5 thermocouples positioned respectively at 10 mm, 20 mm, 30 mm, 70 mm and 100 mm from the fire exposed surface of the slab. These thermocouples measure the temperature increase inside the slab during fire exposure.

The gas temperature inside the oven was controlled by 6 pyrometer plates positioned below the test specimen.

3.1.2 Displacement measurements
The measurement of the vertical displacement of the slab during the test was carried out using three wire displacement sensors attached to the slab at 2600 mm from the wall and at 500 mm, 1000 mm and 1500 mm from the lateral side of the slab (Figure 1). These displacement sensors were attached to a metal beam positioned above the test specimen allowing the measurement of the relative displacement of the slab against the concrete frame.
In addition to displacement sensors, a stereo digital images correlation system positioned above the test specimen was used to measure displacements over the whole test specimen during the fire test.

### 3.2 Test description

The test specimen was positioned on the top of the furnace. The wall and the partition wall of the furnace were positioned at the same level in order to reproduce a fire situation in a real cantilever slab connected to a wall using post-installed rebars. In fact, this wall position allows to avoid the formation of a shadow inside the furnace which may disturb the radiative heat transfer and hence could modify the temperature increase inside the test specimen. The cantilever concrete slab was mechanically loaded by a 325 kg dead weight, centered and positioned at 2200 mm from the wall (Figure 1) and then thermally loaded by ISO fire 834-1 (3) until its failure.

\[
\theta_{\text{gas}}(t) = 20 + 345. \log_{10}(8. t + 1) \tag{3}
\]

Where \( \theta_{\text{gas}} \) is gas temperature in the furnace [K]  
\( t \) is the time of fire exposure [min]

### 4 Results and discussion

#### 4.1 Experimental observations

The slab mechanically loaded has been set as the reference state in the rest of this study. The time \( t=0 \) is considered as the time of the ignition of the furnace burners. Indeed, the mechanical loading of the cantilever slab induced by the dead weight had generated a vertical downward displacement equal to -5.8 mm. This displacement is composed of a mechanical displacement due to the bending of the slab under the loading effect and of a geometrical displacement due to the angle created by the slip of the rebars. Therefore, the vertical displacement of the slab under mechanical load will not be taken into consideration in the rest of this study.

After few minutes of fire exposure, a vertical upward displacement of the slab was observed due to a thermal curvature (Paragraph 4.2). The failure occurred after 1h57min of fire exposure caused by the bond failure and the fall of the slab inside the furnace.

#### 4.2 Thermal analysis

Thermal profiles recorded by the thermocouples inserted inside the wall (Figure 6.a) show a non-uniform distribution of temperature along the chemically-bonded rebars. Indeed, the heating of the slab generated a thermal gradient at the PIRs. The top of the embedded part of the rebars presented the highest temperatures while the end of the embedded part has the lowest temperatures. The maximum temperature was recorded at the top of the PIR by TC1 and was equal to 89°C, reached at the moment of the slab collapse.

The comparison between measured and calculated thermal profiles shows that calculated temperatures are higher than measured temperatures. This difference between thermal profiles can be explained on one hand by the use of the materials thermal properties provided by EC2\(^{10} \) for temperature calculations, which leads to an overestimation of temperatures due to the safety aspect of EC2. On the other hand, this difference may be attributed to the non-consideration of the thermal
bridge coming from the steel rebars prolonged inside the slab (Figure 1) when measuring temperatures with thermocouples. In fact, thermocouples measure only the temperature of the concrete wall, and therefore do not consider the contribution of the steel rebars in the heat transfer. Consequently, the PIR temperatures measured by thermocouples should be underestimated.

Similarly, the calculated temperatures in the slab (Figure 6.b) were higher than temperatures measured using vertical lines of thermocouples due to the use of EC2 parameters in thermal calculations which leads to overestimate the temperature profiles.

![Figure 6: Experimental and numerical temperature profiles. a) At the rebars. b) In the slab](image)

### 4.3 Displacement analysis

Displacements recorded by wire displacement sensors (Figure 7) highlighted a thermal curvature manifested by an upward displacement of the slab since the first minutes of fire exposure. This phenomenon appears due to a differential thermal expansion between the fire exposed and non-exposed surfaces of the concrete slab. Indeed, the concrete directly exposed to fire had expanded under the heat effect, while the concrete non-exposed to fire expanded very little. The difference in thermal expansions had led consequently to the curvature of the slab. As the rotational movements of the slab in the side of the wall were blocked by the chemically-bonded rebars, the free part of the slab has curved and raised upward.

The maximum measured value of vertical displacement due to the thermal curvature was +18.7 mm reached after 28 minutes of exposure to ISO Fire 834-1 ($\theta_{\text{gas}}(28\text{min}) = 832^\circ\text{C}$). Beyond 28 min of fire exposure, the slab started falling slowly inside the furnace. The zero value of vertical displacement was reached again after 92 min of fire exposure. The downward displacement of the slab could be explained by the decrease of the concrete elastic modulus and basically by the progressive decay in the load bearing capacity of post-installed rebars as a result of temperature increase.

Starting from 109 min of fire exposure, the decay in the bearing capacity of the PIRs had become more and more important and the slab failed more quickly inside the furnace until the total bond failure at **117 min** of fire exposure.
Displacements recorded by wire displacement sensors during the fire test showed that displacements measured by the sensor positioned at the mid-width of the slab (sensor 2) were slightly greater than displacements measured by the two lateral sensors which indicated sensitively identical values. These displacement values signify that the concrete slab was curved under the thermal effect in a symmetrical manner to an axis passing through its mid-width plan. This interpretation was confirmed by analyzing the results obtained from the stereo digital images correlation system (DIC) which had shown that under the thermal effect, the vertical displacement fields were concentric, which means that the slab had curved symmetrically in its center (Figure 8).
4.4 Time of collapse predicted by the EAD 330087-00-06.01 method

The design method proposed by the European Assessment Document 330087-00-06.01 for PIRs relies on the same method as the one described in paragraph 2.2. Indeed, the EAD considers that the bond resistance-temperature relationship can be described with an exponential trend curve ($f_{b,m}=a \cdot \exp(-b \cdot \theta)$), to which a temperature reduction factor is applied in order to secure the use of PIRs in fire situation. The bond resistance-temperature curve is cut at 17.2 MPa for a C45/55 concrete, in case the bond resistance exceeds 17.2 MPa (Figure 9). For bond resistance values lower than 17.2 MPa, a proportional reduction factor is applied, justified by the conservative assumption that a PIR cannot present higher performances than a cast-in-rebar (covered in the Eurocode). The method of determining the temperature reduction factor $k_{fi}(\theta)$ is described by equation (4). The variation of $k_{fi}(\theta)$ with temperature is represented in Figure 10.

$$\begin{cases} k_{fi}(\theta) = \frac{f_{b,m}(\theta)}{f_{b,m,req,d}} & \leq 1.0 \text{ for } 20^\circ C \leq \theta \leq \theta_{\text{max}} \\ k_{fi}(\theta) = 0 & \text{else} \end{cases}$$

(4)

Where $k_{fi}(\theta)$ is the temperature reduction factor

$f_{b,m}(\theta)$ is the mean bond resistance at temperature $\theta$

$f_{b,m,req,d}$ is the required bond strength in cold state (= 10 MPa)

$\theta$ is the temperature of the bond

$\theta_{\text{max}}$ is the maximal temperature measured during the test

![Figure 9: Bond resistance-temperature relationship according to the EAD 330087-00-06.01](image1)

![Figure 10: Variation of the $k_{fi}(\theta)$ with temperature](image2)

The design bearing capacity of PIRs in fire situation can be determined from the temperature reduction factor using the characteristic bond resistance at ambient temperature. This characteristic resistance $f_{bd}$ is equal to 4 MPa in the case of C45/55 concrete, multiplied by a safety factor $\eta_{fi}$ equal to 1.5. Thus, the bond resistance of the PIR in fire situation is determined according to the value of the temperature reduction factor, as described in equation (5).

$$f_{bd,fi}(\theta) = \eta_{fi} \cdot k_{fi}(\theta) \cdot f_{bd}$$

(5)

Where $f_{bd,fi}(\theta)$ is the bond resistance corresponding to a bond temperature $\theta$

$f_{bd}$ is the characteristic value of the bond resistance (=4MPa for C45/55 concrete)
\( \eta_{fi} \) is the reduction factor for the design load level for the fire situation (=1.5)

\( k_{\theta}(\theta) \) is the temperature reduction factor

Results show that the calculated load bearing capacity of the PIRs is 4.5 times lower than that obtained by the design method presented in paragraph 2.2. The EAD design method supposes that the structure is not capable to resist to the applied tensile load even at ambient temperature, while the experimental test showed that the structure resisted to 117 minutes of ISO fire exposure. The design of the PIRs according to the EAD\(^7\) method requires a minimum embedment length equal to \( L=225 \) mm to ensure 2 hours of fire resistance under ISO 834-1 fire exposure. However, the design method presented in paragraph 2.2 shows that for an embedment length \( L=225 \) mm, the slab would be able to resist to 210 min of ISO 834-1 fire exposure. Hence, these results highlight that the safety coefficients applied on the EAD design rules are very strict and should be revised.

### 4.5 Failure mode analysis

The fractography analysis showed the presence of a mixed failure mode along the bonded part of the rebars as shown in Figure 11. A resin/concrete interface failure mode was observed at the top of the bonded rebars, up to the two thirds of the embedment length, while the last third of the embedded part of the PIRs presented a steel/resin interface failure mode. This mixed failure mode is the result of a significant thermal gradient present at the PIR at the moment of failure and indicates that the ruin had mainly occurred due to the resin glass transition which led to a significant decay in the load bearing capacity.

![Figure 11: Failure profile observation after the fire test](image)

### 5 Conclusion

This paper presented a full-scale fire test carried out on the CSTB modular gas furnace “Vulcain” in order to validate a fire resistance prediction method for chemically-bonded post-installed rebars. From this study it can be concluded that:

- The proposed method is based on the calculation of the PIRs temperature and on the knowledge of the variation of the bond resistance under the thermal effect.
- The fire test highlighted the presence of a thermal curvature of the slab symmetrically to its mid-width plan and showed that the failure was caused by the slip of the PIRs.
The slab failure occurred after 1h 57 minutes of ISO Fire 834-1 exposure, i.e. only 3 minutes before the fire resistance duration predicted by the model. This result confirm the accuracy of the calculation method in predicting the fire resistance duration of post-installed rebars and shows that the established assumptions are in good agreement with the reality. The comparison with the design method proposed by the EAD 330087-00-06.01 showed that the EAD largely underestimates the fire resistance of PIRs. Therefore, the used safety factors are very strict and should be revised.

The fractography showed the presence of mixed failure mode along the PIR indicating that the resin glass transition was responsible for the slab collapse.

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CONNECTIONS UNDER SPECIAL LOADINGS:
SEISMIC / IMPACT
A REVIEW OF EXISTING PROVISIONS FOR SEISMIC QUALIFICATION AND DESIGN OF POST-INSTALLED FASTENERS

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ABSTRACT

In the last decade many efforts were put to define both qualification procedures and design procedures for post-installed fasteners under seismic actions both in Europe and in the US.

However, significant differences may be found in the two approaches, both in terms of testing protocols and relevant acceptance criteria and in design criteria, which may generate confusion for practitioners in defining design requirements.

A general review of existing design criteria for post-installed steel-to-concrete connections under seismic action is presented, with particular emphasis on the new approach of Eurocode 2 part 4.

Design options by protection of the fastener or by protection of the attached element are introduced, discussing the capabilities and limitations of each solution.

On the basis of the introduced design criteria, requirements for qualification are presented and discussed. For such design criteria it is shown how a qualification procedure should account for both load and crack cycling to guarantee suitability for both structural and non-structural applications.

1 Introduction

Post-installed fastening to reinforced concrete members is of crucial importance for the mechanical resistance and stability of both structural and non-structural members.

When connecting structural elements, the fastening should behave in such a way as to adequately transfer the internal forces from the connected element to the concrete members. Typical applications include the connection of steel beams to concrete cores and walls or of steel columns to concrete foundations. Additionally, the fastening may be required to accommodate relative rotation between the connected elements or parts of them. As will be discussed, the need to satisfy this requirement or not may lead to totally different design options.

Qualification procedure, according current European guideline1, assess the behavior of single anchor subjected to load cycles and crack movement separately. With reference to qualifications results...
carried out for post-installed mechanical it will be shown how crack movement test is the most demanding and definitely decisive in seismic evaluation with respect to load cycling test.

2 Design criteria

Seismic actions affect anchors in two different ways. First, they induce cracking and crack cycling in the primary structure, and, second, the movement of the structure generates dynamic tension and shear forces on anchors.

As response to earthquake ground motion, the entire structure experiences displacements and consequently deformation of its members. In reinforced concrete (R/C) members a proper design allows to localize inelastic deformations in specific regions ensuring inelastic rotation capacity (i.e. the plastic hinges [PH]). However, the large scatter in the values of crack width and the unpredictable location of cracks in these regions make them unreliable for fastening. Additionally, depending on the seismic role of the connected elements (primary, secondary or non-structural) an appropriate behavior model of the connection in the global structural analysis may be required. It is consequently necessary to operate a differentiation among several types of connections.

As effect of seismic action not only the base material experiences cracking and crack cycling but also dynamic forces are generated in the attached element and, as a consequence, in fasteners. Thus response will depend on the combination of both.

For sake of simplicity, consider such sources of loads as a periodic function: the response of fastener will depend on amplitude number of cycles. Moreover, time histories (i.e. cyclic crack, and cyclic load) may be in phase or out of phase leading a different effect in fastener’ response.

Capacity design in frame structure under seismic loads applies in case of a steel beam connected to a R/C element. The requirement in case of frame bending behavior regards the well known “weak beam/strong column” approach.

The required plastic rotation $\theta_p$ can be achieved by yielding of the attached element (Figure 1 - a) or of the fasteners (Figure 1 – b). In the first case there is “protection of the fastener”, while the second case corresponds to “protection of the attached element”. For the latter it is immediately evident how the single fastener is required to undergo a significant elongation to achieve the required global rotation, which will correspond to a specific requirement on its stretch length.

prEN 1992-4 “de facto” limits the design option that account for fasteners’ ductility in case of connections of secondary members only (i.e. secondary members). Connections between primary members are restricted to “fastener protection” since the contribution of fasteners in structural energy dissipation is difficult to be determined, and definitely excluded.

Connections of a non-structural component can then be designed according to both approaches without significant limitations since they do not influence the response of the primary structure. However, in critical regions in R/C structures uneven distribution of large cracks due to yielding of reinforcement and concrete spalling may occur. Consequently, all the existing codes prevent fastening in plastic hinge zones.
Figure 1 Plastic hinge: (a) reinforced concrete member; (b) steel member; (c) yielding of the attached element; (d) yielding of the fastener

Additionally, prEN 1992-4\(^4\) also accounts, independently on the design approach, for the hammering effects due to the annular gap between fastener and fixture in case of shear loading by introducing a reduction factor on the resistance side (only for reasons of simplicity this effect is not accounted for by an amplifying factor on the action side). This factor reduces by half the fastener resistance for connections with hole clearance while it is equal to one if no hole clearance between fastener and fixture is provided.

2.1 Design by protection of fastener

When yielding of the attached element is sought, yielding of the fasteners has automatically to be prevented. The fasteners are consequently designed with no ductility requirement as non-dissipative elements that do not contribute to the overall ductile behavior of the structure. While ACI 318\(^3\) allows this option only if the anchor cannot meet specific ductility requirements, CEN/TS\(^6\) considers design by protection of the fastener as a standard design option. However, both codes account for:

- elastic design: the fastening is designed for the maximum action obtained from the design load combinations that include seismic actions assuming elastic behavior of the fastening and of the structure;
- capacity design: the fastening is designed for the maximum action that can be transmitted to the fastening based either on the development of a ductile yield mechanism in the attached steel component (Figure 2 - a) or in the steel base plate (Figure 2 - b) taking into account overstrength effects, or on the capacity of a non-yielding attachment (Figure 2 - c).
In particular, it is generally prescribed that the design strength of the part of the connection supposed non-yielding exceeds a multiple of the nominal strength of the attachment. The multiple account for both material overstrength and strain hardening.

In case of flexible base plate until yielding, prEN 1992-4[^4] warns against assuming a plastic hinge in the fixture because of the difficulties in accounting both for the redistribution of loads to the individual fasteners of a group and for the low cycle fatigue behavior of the fixture.

With this regard, it is worth noticing that, if the plastic hinge is in the fixture but outside the anchor rows, the derivation of forces on the single fasteners can still be carried out according to ordinary linear elastic methods described in the codes. As matter of fact, capacity design applied to connections with fasteners, implicitly needs to maintain rigid base plate also in inelastic regime, except for special cases as mentioned before.

Finally, if the fastener is not supposed to yield and an overstrength factor is accounted, the maximum force acting on the fastener will be significantly lower than its resistance.

In the spirit of capacity design, consider for instance, a beam to column connection. For a plastic hinge formation in the attached element, relationships holds between the design values of the moment of resistance of the column $M_{c,Rd}$ and of the beam $M_{b,Rd}$:

$$M_{c,Rd} = \gamma_{Rd} \cdot M_{b,Rd}$$

(1)

where $\gamma_{Rd}$ accounts for material overstrength. Being the moment of resistance of the fastening $M_{f,Rd}$ equal to $M_{c,Rd}$ and considering a linear relationship between the bending moment acting on the group and the maximum tensile force acting on a single fastener (e.g. true in case of rigid base plate only), the maximum value of the design tensile force on the fastener $N_{E,d}$ for the seismic design situation is:

$$N_{E,d} = \frac{N_{f,Rd}}{\gamma_{Rd}}$$

(2)

where $N_{f,Rd}$ is the design value of the fastener resistance for monotonic loading. Adopting a value of $\gamma_{Rd}$ equal to 1.3 and assuming a decisive failure mode in steel with a coefficient of variation of the ultimate load not greater than 15% (as allowed for reference/service condition tests by qualification procedures[^1],[^7],[^8]), Eq. 2 turns into:
Giovanni Muciaccia and Angelo Marchisella

\[ N_{Ed} \leq \left( \frac{N_{f,Rk}}{\gamma_s \cdot \gamma_{Rd}} \right) = \frac{(1 - 1.645 \cdot 0.15)}{(1.15 \cdot 1.3)} \cdot N_{f,Rm} = 0.50 \cdot N_{f,Rm} \]  

(3)

where \( N_{f,Rk} \) and \( N_{f,Rm} \) are the characteristic and mean resistance, respectively, of the fastener for monotonic loading and \( \gamma_s \) is the material partial safety factor.

It can then be concluded that, when adopting a capacity design approach, the maximum value of the tensile force acting on a single anchor during a seismic event should be limited to the 50% of its capacity for monotonic loading.

### 2.2 Design by protection of attached element

When protection of the attached element is required, the fastening is expected to fail by ductile steel yielding. All the codes\(^3,6,1\) require the following conditions to be satisfied:

- the tensile steel capacity of the fastening needs to be smaller than the tensile capacity governed by brittle failure modes (concrete cone breakout or pullout) as steel yields well before the concrete anchorage fails;
- sufficient elongation capacity of the fasteners is required so that the displacement associated with the design-basis earthquake can be achieved.

ACI 318\(^3\) introduces the stretch length as the place where inelastic deformations of the fasteners should develop. Prescription of eight diameters as stretch length is given for anchors which incorporate a reduced section over their length (e.g. threaded). Yielding has to occur over the unreduced portion of the bolt (e.g. unthreaded) within the stretch length. Buckling, experienced in load reversal, should be definitely prevented by reducing the effective-length with constraints.

However, the requirement on the stretch length only assures that single fasteners are able to develop a given displacement (which will also depend on the ductility of the steel in the portion of the fastener which is elongating) but it provides no assessment of the rotational capacity of the connection. Consequently, prEN 1992-4\(^4\) explicitly introduced the rotation capacity of the connection \( \delta_p \) as:

\[ \delta_p = \frac{\delta_N}{s_{max}} \]  

(4)

where \( \delta_N \) is the ductile displacement of the anchor and \( s_{max} \) is the distance between the row of anchors undergoing ductile yielding and the opposite edge of the baseplate (see Figure 3).

Overstrength approach (i.e. capacity design) in prEN 1992-4\(^4\), in case of tensile force, leads the following inequality:

\[ N_{Rk,s} \leq 0.7 \cdot N_{Rk,conc} / \gamma_{inst} \]  

(1)

where \( N_{Rk,s} \) and \( N_{Rk,conc} \) are the characteristic resistance for steel failure and the minimum characteristic seismic resistance for all concrete related failure modes (including combined pull-out and concrete, concrete blow out, splitting), respectively, and \( \gamma_{inst} \) is a factor accounting for the sensitivity to installation conditions (ACI 318\(^3\) implicitly accounts for similar provisions).
3 Qualification

Evaluation of seismic performance for anchors is treated in ETAG 001\textsuperscript{1}. Two seismic performance are distinguished: C1 and C2 being C2 the most demanding.

Table 1 synthetizes tests typologies. As a matter of fact C1 category definitely reflects ACI\textsuperscript{7,8} provisions about seismic qualification of anchors. C1 and C2 categories differ in number and sequence of cycles (stepwise decreasing or increasing respectively) with respect to simulated seismic load cycling test, even though there are not substantial differences in terms anchor performances, as proved by Hoehler\textsuperscript{9}. Moreover, step wise increase allows to experience a stiffness change comparable to monotonic reference. Category C2 finally introduces a specific test with constant tension load under crack movement, i.e. C25. Crack opening increases progressively until 0.8 mm while imposed axial load switches from 0.4 $N_{um,C21}$ to 0.5$N_{um,C21}$ hereafter 0.5 mm cycles are concluded.

Assessment requirements, in case of C2 category, regard: displacement during cyclic test ($< 7$ mm), coefficient of variation for ultimate loads and displacements at peak for monotonic tests. Seismic performance is condensed in the $\alpha$ reduction factor which is taken $\alpha_{N,seism} = \alpha_{C21} \cdot \min (\alpha_{C23}; \alpha_{C25})$ and $\alpha_{V,seism} = \alpha_{C22} \cdot \alpha_{C24}$ for tension and shear respectively. The characteristic resistance for pullout is:

$$N_{Rk,seism} = \alpha_{N,seis} \cdot \beta_{cv,N} \cdot N_{Rk,0} \quad (2)$$

where $N_{Rk,0}$ is the characteristic value for pullout in cracked concrete, $\beta_{cv,N}$ accounts for scatter in results and it is not substantial in the aim of the present discussion.

Experience over the last years revealed crack movement test as the most demanding and typically decisive in the assessment. To prove it, results for post -installed expansion anchors (sleeve and wedge type) are reported. Thirty-two seismic assessments are considered with reference to displacement at Damage Limit State (DLS) and reduction factors for C25 and C23 test according ETAG 001\textsuperscript{1}. Considered diameters are in range 6 mm ÷ 24 mm.
### Table 1 Overview of seismic qualification according to ETAG 001\(^1\)

<table>
<thead>
<tr>
<th>Name</th>
<th>Test Description</th>
<th>Load(^{(1)})</th>
<th>(\Delta w) (^{(2)})</th>
<th>Steps(^{(3)})</th>
<th>Red.Factor (\alpha) (^{(4)})</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Category C1</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C1.1 Pulsating tension load</td>
<td>(N_{eq} = 0.5 \cdot N_{um})</td>
<td>0.5</td>
<td>140</td>
<td></td>
<td>(N_{red} / N_{eq})</td>
</tr>
<tr>
<td>C1.2 Alternate shear load</td>
<td>(V_{eq} = 0.5 \cdot V_{um})</td>
<td>0.5</td>
<td>140</td>
<td></td>
<td>(V_{red} / V_{eq})</td>
</tr>
<tr>
<td><strong>Category C2</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C2.1 Tension with static crack at failure</td>
<td>0.8</td>
<td></td>
<td></td>
<td></td>
<td>(N_{umC21} / N_{um})</td>
</tr>
<tr>
<td>C2.2 Shear with static crack at failure</td>
<td>0.8</td>
<td></td>
<td></td>
<td></td>
<td>(V_{umC22} / 0.8V_{um})</td>
</tr>
<tr>
<td>C2.3 Pulsating tension load</td>
<td>(N_{\text{max}} = 0.75 \cdot N_{um,\text{C21}})</td>
<td>0.5+0.8</td>
<td>75</td>
<td></td>
<td>(N_{red} / N_{\text{max}})</td>
</tr>
<tr>
<td>C2.4 Alternate shear load</td>
<td>(V_{\text{max}} = 0.85 \cdot V_{um,\text{C22}})</td>
<td>0.5+0.8</td>
<td>75</td>
<td></td>
<td>(V_{red} / V_{\text{max}})</td>
</tr>
<tr>
<td>C2.5 Cyclic crack opening</td>
<td>(N_{w2} = 0.5 \cdot N_{um,C21})</td>
<td>0.8</td>
<td>59</td>
<td></td>
<td>(N_{red} / N_{w2})</td>
</tr>
</tbody>
</table>

\(^{(1)}\) Maximum applied load during cycles; “um” subscript intends the mean value in reference cracked concrete

\(^{(2)}\) Maximum value for crack opening during cycles, values are in mm

\(^{(3)}\) Total number of steps involved in test

\(^{(4)}\) “red” subscript intends the reduced load to complete a test satisfying all the assessment requirement

In Figure 4 reduction factors are shown for crack movement test (C25) and for cyclic tension test (C23); unitary value for threshold means no reduction with respect to reference monotonic test with 0.8 mm crack opening (C21). If often happens that \(\alpha_{c21}\) is equal to 1, while \(\alpha_{c25}\) is always lower. Relevant displacements for DLS are reported in Figure 5. C25 and C23 tests share the same requirement, i.e. displacement has to be less than 7 mm considering the average value for the test series. However, in case of crack movement, anchor slip increases due to the severe condition of damage in bearing zone as a consequence of alternate tension and compression. Sleeve anchors displace more with respect to wedge type, nevertheless influence of specific product (i.e. cone/sleeve friction coefficient, surface treatment) cannot be excluded, hence the trend has to be considered qualitative.

In Figure 6 ratios for displacement at DLS and reduction factors are represented. In no case C25 displacement at DLS is lower than C23 displacement; the same applies for reduction factors except for one case. C25 test can be definitely considered decisive in the qualification process, i.e. the associate reduction factor and displacement at DLS governs the declared performances for the anchor under seismic loads.

Concluding, crack movement test is decisive for all the cases in establishing the reduction factor \(\alpha_{\text{seis}}\) for pullout capacity and displacement at DLS. Results of cyclic load tests are not conservative at all. Hence, to date, seems to be a great disproportion in qualification of post-installed anchors looking at Europe\(^1\) and US\(^7,8\) regulations.
Figure 4: Reduction factors for expansion anchors for C25 (left) for C23 (right)

Figure 5: Target displacement at DLS for cyclic test for C25 (left) for C23 (right)
4 Conclusion

Design criteria for post-installed steel-to-concrete connections under seismic action are presented, also detailing and commenting the provisions of existing design codes both in US and in Europe. It is shown how capacity design can be efforted, depending both on the intended use of the connection (structural or non-structural) and on the adopted design criterion (protection of the fastener or of the attached element). Current seismic qualification procedures account both for cycling load and crack movement test. Nevertheless, crack movement prevails in the assessment, questioning the opportunity to overlook the cyclic load tests in case of tensile behavior.

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EXPERIMENTAL BEHAVIOR OF REAL-SIZE POST-INSTALLED STEEL-TO-CONCRETE CONNECTIONS UNDER SEISMIC ACTION

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ABSTRACT

Connecting steel members to concrete elements by post-installed fasteners represents a frequent solution in common practice. In recent years, several design codes (ACI 318, Eurocode 2) introduced specific design criteria for steel-to-concrete connections under seismic action.

However, despite a significant number of tests carried out on single fasteners, a reduced effort was dedicated to investigate the behavior of the whole connection.

An experimental investigation on real size groups of six post-installed fasteners located far from free edges is presented. Several design options are investigated, according to protection of the fastener or protection of the attached element criteria. The effects of the stretch length and of the stiffness of the attached element are also investigated and discussed.

For connections designed by protection of the fastener it is found that no significant difference is detected with respect to established behavior for standard steel-to-steel connection when capacity design criteria are adopted.

On the other hand, in connections designed by protection of the attached, not only a proper stretch length is necessary to provide sufficient rotational capacity of the connection, but specific solutions are also required to prevent cumulating unrecoverable displacements at load reversal.

1 Introduction

Under seismic action, fasteners are expected to transfer cyclic actions from the member to be connected to the concrete as reliably as possible. Their behavior, however, is still not completely understood in seismic conditions. During the last years, a number of scientific contributions have been devoted to investigating what should be the most appropriate experimental technique to assess the behavior of the single anchor1-5, while less attention was dedicated to the behavior of the entire connection.
2 Experimental investigation

Design criteria for post-installed fasteners under seismic actions were extensively discussed during the last years, while experimental results are generally available for single fasteners\textsuperscript{2,3,4} only and not for fastener groups.

An experimental investigation was carried out on real size post-installed fastener groups designed according to both protection of the fastening or protection of the attached element criteria installed far from free edges in low strength (C20/25) uncracked concrete.

The two design criteria are discussed in detail in Muciaccia\textsuperscript{6}. Synthetically, in the protection of the fastening case, the connection is designed for the maximum capacity of the attached element (also accounting for a proper overstrength factor), while, when protection of the attached element is required, the fastening is expected to fail by ductile steel yielding.

Anchor groups were made of six anchors qualified according to ETAG 001 C2 category C2\textsuperscript{7}, installed in an unreinforced concrete slab which was clamped to a strong floor (Figure 1 - a). The type of fastener and the geometry of the fastening vary according to the desired design option, as it will be detailed hereinafter.

Each anchor group was subjected to alternate shear and bending moment. A transversal alternate load was applied to the attached element. The distance between the direction of application of the load and the concrete surface was selected in order to apply the desired pair of shear and bending moment to the fastening. The lever arm between the actuator and the concrete surface is equal to 150cm. The 100 kN hydraulic jack used to apply the load was hinged to the top of the attached element (Figure 1 - b).
2.1 Test protocol

Tests were displacement controlled choosing as feedback parameter the displacement applied at the top of the attached element, for which the time history was selected according to the ECCS recommendations\textsuperscript{15}.

The applied alternate displacement and the number of cycles is reported in Table 1, where \( e_y \) is the displacement corresponding to the yielding of the first component (which may occur in the attached element or in the fasteners, depending on the design option) and it was previously determined by a monotonic test.

<table>
<thead>
<tr>
<th>Step</th>
<th>Range</th>
<th># of cycles</th>
<th>Step</th>
<th>Range</th>
<th># of cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>((-e_y/4) ÷ +e_y/4)</td>
<td>1</td>
<td>4</td>
<td>((-e_y) ÷ +e_y)</td>
<td>1</td>
</tr>
<tr>
<td>2</td>
<td>((-2e_y/4) ÷ +2e_y/4)</td>
<td>1</td>
<td>5</td>
<td>((-2e_y) ÷ +2e_y)</td>
<td>3</td>
</tr>
<tr>
<td>3</td>
<td>((-3e_y/4) ÷ +3e_y/4)</td>
<td>1</td>
<td>6</td>
<td>((-3e_y) ÷ +3e_y)</td>
<td>3</td>
</tr>
</tbody>
</table>

3 Results and discussion

3.1 Protection of the fastening case

The geometry of the fastening and the details of the arrangement of displacement transducers are reported in Figure 2. The fasteners consisted in six M16 wedge anchors with an embedment depth of 85mm.

![Figure 2 Protection of the fastening - geometry of the connection (a) and arrangement of displacement transducers](image)

The fastening is designed such that the four anchors in the corner alternatively react against the tensile action induced by bending and the remaining two fasteners entirely take up the applied shear.
The bending moment acting on the connection $M_{c,Rd}$ is calculated according to Equation 1, where $M_{a,Rd}$ is equal to moment of resistance of the attached element and $\gamma_{Rd}$ is the overstrength factor, taken an equal to 1.3.

$$M_{c,Rd} = \gamma_{Rd} \cdot M_{a,Rd}$$

(1)

The nut was removed from the two central fasteners after the anchor installation to prevent them from being loaded in tension and thus to ensure the desired transmission of shear. Additionally, to ensure that shear load could not be initially transferred to the fasteners located in the corners, a malleable lead layer was interposed between the bolt and the hole in the fixture filling the gap.

As shown in Figure 2–b, the displacement of each of the four anchors in the row was continuously monitored during testing by four 50mm LVD Transducers (positions 1, 2, 3, 4), while the global displacement of the base plate in the load direction was monitored by two 10mm LVD Transducers (positions 5, 6). The convention for the load sign is such that the load is assumed positive when inducing tension in positions (2, 3) and negative when inducing tension in positions (1, 4).

During the test no failure of any fasteners was observed; yielding in the attached element was correctly induced, with a typical dissipative behavior, as reported in Figure 3 – a. Shear connectors were able to take up the base shear showing a reduced in-plane rotation (Figure 3 – b), which reached 3.6 mrad only.

Figure 4 shows the displacement recorded at positions 1 and 4 during the load application (only the relevant portion of loading is reported). The same behavior was detected for fasteners at positions 2 and 3. It can be noticed how the displacement of the fasteners increases during the load application and it is partially recovered at load reversal. An unrecoverable displacement is detected at the end of cycles (equal to 0.63mm at position 4) without affecting the global hysteretic behavior of the attached element.

![Figure 3](image.png)

Figure 3 Protection of the fastening - base shear vs top displacement (a) and base fixture displacement vs time (b)
3.2 Protection of the attached element case

According to prEN 1992-4\textsuperscript{8}, the fastening is expected to fail by ductile steel yielding if the tensile steel capacity of the fastening is smaller than the tensile capacity governed by brittle failure modes (concrete cone breakout or pull-out) as steel yields well before the concrete anchorage fails. Such condition is assured if the following condition is satisfied.

\[ N_{Rk,s} \leq 0.7 \cdot N_{Rk,conc} / \gamma_{\text{inst}} \]  

where \( N_{Rk,s} \) and \( N_{Rk,conc} \) are the characteristic resistance for steel failure and the minimum characteristic seismic resistance for all concrete related failure modes (including combined pull-out and concrete, concrete blow out, splitting), respectively, and \( \gamma_{\text{inst}} \) is a factor accounting for the sensitivity to installation conditions.

The geometry of the connections and the positions of displacement transducers is reported in Figure 5. To achieve ductile steel yielding it was not possible to satisfy the hierarchy given by Equation 2 by adopting a mechanical fastener. Consequently, an adhesive anchor with 8.8 class threaded rod as insert was selected. The embedment length was calculated according to Equation 2 and it resulted equal to 310mm. To ensure failure in the fastening, the size (IPE 240) and steel class (S275) of the attached element were selected such that its resisting moment was higher than the one provided by the connection. Finally, the size of the shear connectors was increased to M30 to assure that failure in tension induced by bending in the corner anchors precedes shear failure of the shear connectors.
A sufficient elongation capacity of the fasteners so that the displacement associated with the design-basis earthquake can be achieved is also required by prEN1992-4. Such condition is achieved by requiring a minimum stretch length equal to eight times the anchor diameter and, for anchors which incorporate a reduced section over their length (e.g. threaded, as in the present case), by additionally ensuring that yielding occurs over the unreduced portion of the bolt (e.g. unthreaded) within the stretch length prior to failure in the reduced section.

Consequently, in the investigated case, the stretch length is equal to 130mm (higher than eight times the diameter). This was achieved by designing a specific anchor chair (Figure 6). Additionally, to prevent unrecoverable displacement during load reversal, the top end of the fastener was clamped on both sides to the top plate of the anchor chair. No failure was detected during the load cycling. After the last cycle the load was monotonically increased up to failure, which consisted in steel failure of fastener at location 1.
Displacement cumulated during cycles was almost entirely recovered during load reversal, as shown in Figure 7 (most likely displacement corresponding to bar slip at the concrete surface was not recovered), such that the residual displacement has a negligible influence on the global hysteretic behavior of the connection. Finally, the experimental base shear vs curvature relationship for the connection can be derived, as reported in Figure 8. A behavior resembling a typical hysteretic behavior of a dissipative steel-to-steel connection is noticed, with high values of rotational capacity achieved (more than 40 mrad), even higher than the ones typically required to connections in steel buildings (EN1992-8 requires 35 mrad for structures of high ductility class).

Figure 7 Protection of the attached element- base shear vs top displacement (a) and fasteners displacement (b)

Figure 8 Protection of the attached element - base shear vs curvature
3.3 Discussion

The presented cases were selected such to be representative of design by protection of the fastening - by inducing yielding in the attached element - and design by protection of the attached element – by inducing yielding in the fasteners.

It is shown that, where the performances of the connection in terms of energy dissipation and rotation capacity are governed by the attached element, the behavior of a steel-to-concrete connection designed according to capacity design rules shows no difference with respect to the behavior of standard steel-to-steel connection.

Following the criterion of protection of the attached element by introducing an anchor chair, it is possible to reach performances similar to the previous case, achieving a rotational capacity even higher than the one typically required for connections in steel buildings. An adequate stretch length and a bidirectional clamping of the fasteners in tension to avoid unrecoverable displacement are required to achieve such result.

In both cases, the nut was removed from all the fasteners located along the center line of the baseplate to ensure they would entirely take up the shear load.

In the second case higher values of bending capacity with respect to the previous solution can be achieved since the fasteners are fully utilized. Moreover, any profile with a resisting moment higher than the one provided by the connection is suitable. This implies that the maximum size of the element to be connected is limited only by the dimensions of the base plate, which is relevant in practice where dimensions of both steel profile and baseplate represent boundary conditions for the design of the connection.

Shear action to be transferred also increase, which requires an increase in size of shear connectors (from M16 to M30 for the investigated cases).

It may then be concluded that no additional restriction in terms of application needs to be introduced for such designed connections.

4 Conclusion

Behavior of fastening (i.e. the assembly of the fasteners and of the fixture) subjected to alternate shear is experimentally investigated in two different cases, designed to achieve, under seismic action, protection of the fastener or protection of the attached element, respectively.

For connections designed by protection of the fastener it is found that no significant difference is detected with respect to established behavior for standard steel-to-steel connection if capacity design criteria are adopted. This suggests that the main structure may be designed according to existing criteria for capacity design, developing the required rotation in the attached element.

Differently, the introduction of a proper stretch length is necessary to develop the required elongation and, finally, to provide sufficient rotational capacity for connections designed by protection of the attached element criterion. Specific solutions to prevent cumulating unrecoverable displacements at load reversal are also required.
When such conditions are satisfied, no additional restriction in terms of application needs to be introduced. However, their use in connecting primary seismic members in moment resisting frames still requires extensive numerical simulations to assess their effective contribution to the energy dissipation in a global structural analysis. In the future, additional experimental research may also be carried out to investigate different configurations (in terms of number of arrangement of fasteners), the presence of cracks in the base material, the effect of close edges or the use of alternative solutions to carry shear load (e.g. shear lags).

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INFLUENCE OF IMPACT PRELOADING ON THE RESIDUAL CONCRETE CONE CAPACITY

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ABSTRACT

Due to car collisions or high ground accelerations caused by earthquakes or nuclear explosions, fasteners as a part of whole may be subjected to increased loading rates over their service life. There are basically two scenarios under such loading action: (1) immediate failure of the fastener or (2) uncertain damage caused by the impact load. This latter case is of particular interest in the fastening technology, because fasteners after impact loading actions may be used further with uncertain safety. For the safe operation of fasteners under such loading conditions, a pre-qualification procedure concerning displacement criteria for impact-loaded anchors was required. However, when assessing the structural safety of the anchors, not only displacement criteria according to ACLS 97101, but also the residual capacity - as an important indicator of the degradation - must be taken into account. When concrete breakout is the decisive failure mode, it is assumed that the impact pre-load leads to crack initiation in the concrete matrix and the resulting micro-cracks may lead to the deflection of the concrete cone capacity during the second loading (residual concrete cone capacity). The available studies on the rate dependent behavior of concrete emphasize only the influence of loading rate on the ultimate capacity, crack pattern, failure mode of concrete. However, the influence of the impact preloading on the residual concrete cone capacity of fasteners was not investigated yet. In this recent study, numerical and experimental investigations were carried out to account for the damage caused by impact preloads on fasteners. The numerical investigations were carried out using the experimentally verified M2-O Microplane Model, which accounts for the rate dependent concrete behavior. The results of the numerical analysis show the almost linear relationship between the level of the impact pre-load and the residual concrete cone capacity. Furthermore, the loading rate of the applied pre-load has a significant influence on the degradation. For the experimental verification, a new testing device was designed, with which impact loads up to 100 MN/s loading rate could be induced on fasteners. The first test results are found to be in a good agreement with the numerical results and the detrimental effect of the applied impact pre-load on the residual concrete cone capacity was verified.

1 Introduction

The concrete behavior including the ultimate resistance, failure mode and crack pattern changes with increasing loading rate. Three effects govern the rate dependent behavior of concrete: (1) creep of bulk material between the opening cracks, (2) rate dependency related to the crack initiation, (3) the
activation of structural inertia forces. The first effect dominates at very small loading rates i.e. at sustained loads; the second and the third effects dominate at increased and very high loading rates (impact load). However, in the latter case, the rate dependency cannot be neglected. The rate dependent concrete behavior also applies for fasteners, for instance, when concrete cone under tension is the decisive failure mode. The concrete cone capacity of cast-in headed studs was investigated numerically by Ožbolt et al. using the rate sensitive M2-O Microplane Model. The dynamic analysis was carried out on anchors with effective embedment depth of $h_{ef}=150$ mm, $h_{ef}=889$ mm and $h_{ef}=1500$ mm and the calculated relative resistance of the concrete cone is shown Figure 1a. The significant increase in the ultimate concrete cone capacity is attributed to the structural inertia forces at loading rate higher than approximately 20,000 mm/s. Additionally, the size effect was also investigated, which showed that for larger specimens, due to more structural inertia, a bigger influence of the loading rate was observed. Although the rate dependent M2-O Microplane Model was experimentally verified on L-Specimens and on compact tension specimens, the experimental verification of the numerically obtained results on concrete cone is not practically possible due to the requirement of very high loads (at high loading rates) and extremely big concrete specimens. Results of unconfined impact tension loading tests on cast-in headed studs show the influence of loading rate on the ultimate concrete cone resistance (Figure 1b). However, these impact loading tests were carried out on cast-in headed studs with relatively small embedment depth of $h_{ef}=40$ mm and loading rates higher than $4 \times 10^4$ kN/s could not be induced using servo-controlled hydraulic test apparatus. Therefore, only a qualitative comparison of the experimental and numerical results was possible. During the impact loading tests of Fujikake, an average increase in the tension resistance of about 60% was observed at approx. 40,000 kN/s loading rate.

![Figure 1: a) Influence of loading rate on the ultimate concrete cone resistance, numerical investigations (Ožbolt et. al. 2006); b) Influence of loading rate on the ultimate concrete cone resistance, experimental investigations (Fujikake et. al. 2003)](image_url)

The existing studies on the rate dependent concrete behavior emphasize the increase of the ultimate resistance at increased loading rates (both for rate dependent concrete cone behavior as well as the behavior of compact tension and L-specimens) and the results are valid only for the resistance against the first impact. In the fastening technology, fasteners may be subjected deliberately or accidently to such impact loading actions, which does not lead to the immediate failure of the anchor. If no visible damage can be seen on the concrete surface, the fasteners are commonly used further without retrofitting and most importantly, with uncertain safety. Due to an applied impact preload,
which does not lead to failure, results in crack initiation in the cement matrix and the degradation due to this damage can be quantified e.g. by the assessment of the residual concrete cone capacity. Since no investigations regarding this damage phenomenon exist, further research was necessary.

2 Scope

Within the scope of this recent study, numerical and experimental investigations were carried out with particular interest on the effect of impact preloading on the residual concrete cone capacity of the fasteners. For the numerical investigations, 3D finite element model of headed studs was made and the dynamic analysis was carried out using the rate dependent M2-O Microplane Model\(^2\). The basic principles of the used version of the Microplane Model are presented briefly in the following paragraph. After the preliminary calculations, impact loading (until failure) and impact pre-loading followed by residual pullout tests were performed on headed studs and on bonded anchors to verify the numerically obtained results. To this end, a new test arrangement was designed at Institute of Construction Materials, with which impact loads up to 100 kN could be induced and the loading rate of 100,000 kN/s could be exceeded.

3 Numerical investigations

3.1 3D Finite Element Model

For the numerical investigations of the concrete cone failure, two separate 3D finite element models of headed studs were made with \(h_{ef}=50\) mm and \(h_{ef}=70\) mm embedment depth using the Software FEMAP® (Siemens). Based on the double symmetric geometry and boundary conditions, the discretization of a quarter model was sufficient. The size of the concrete block was selected such that the formation of the undisturbed full-size concrete cone could be ensured. The concrete block was meshed with 4-node tetrahedral elements and the steel was meshed using 8-node hexahedral elements. Furthermore, a contact layer was modelled between the anchor shank and the concrete, in which 1D bar elements were applied. The bar elements could only bear compressive forces, therefore no tension forces could be transferred at the interface between the steel and the concrete.

![3D finite element quarter model of a headed stud with boundary conditions](image)

The nodes located on the quarter circular curve on the upper concrete surface were constrained in the vertical direction and all the nodes located on the two inner faces were constrained in X and Y directions, respectively. The geometry data was exported to MASA\(^10\) (Microscopic Space Analysis),
and the numerical analysis was performed using the above mentioned M2-O Microplane Model, which accounts for the rate dependent concrete behavior as well as for the damage process according to the cyclic law.

Table 1: Overview of the used material properties

<table>
<thead>
<tr>
<th>Material Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cyl. Compressive strength, $f_c$ (N/mm$^2$)</td>
<td>42,40</td>
</tr>
<tr>
<td>Tensile strength $f_t$ (N/mm$^2$)</td>
<td>2,90</td>
</tr>
<tr>
<td>Young’s modulus, $E$ (N/mm$^2$)</td>
<td>30.800</td>
</tr>
<tr>
<td>Fracture energy, $G_f$ (N/m)</td>
<td>0,096</td>
</tr>
<tr>
<td>Poisson’s ratio, $v_c$</td>
<td>0,18</td>
</tr>
<tr>
<td>Concrete Weight density (kg/m$^3$)</td>
<td>2.400</td>
</tr>
<tr>
<td>Steel (linear elastic), $E$ [GPa]</td>
<td>210</td>
</tr>
</tbody>
</table>

3.2 M2-O Microplane Model

The rate dependency was introduced in the M2-O Microplane Model in 2006\(^2\) and its general applicability for the realistic prediction of the rate dependent concrete behavior numerically was verified through different studies\(^4,7\). The M2-O Microplane Model accounts for two effects: (1) rate dependency related to the initiation of the micro-cracks and (2) creep of the bulk material between the opening micro-cracks. The rate dependency can be calculated for each microplane stress-strain component according to Equation 1\(^2\).

$$\sigma_M(\varepsilon_M) = \sigma^0_M(\varepsilon_M) \left[1 + c_2 \ln \left( \frac{\dot{\gamma}}{c_1} \right) \right]$$

with

$$\dot{\gamma} = \sqrt{\frac{1}{2} \dot{\varepsilon}_{ij} \dot{\varepsilon}_{ij}}$$

and

$$c_1 = \frac{c_3}{s_{cr}}$$

where $c_2$ and $c_3$ are material constants, which were determined by fitting test data on uniaxial compressive tests from Dilger et. al.\(^2\) and $s_{cr}$ is the assumed spacing of the developing micro-cracks.

In the 3D dynamic analysis, the following Equation is solved, which calculates the system of unknown displacements in each time step (Voigt notation):

$$M \ddot{u}(t) + C \dot{u}(t) - f(t) = 0$$

(2)

Where $M$= Mass matrix, $D$= damping matrix, $\ddot{u}$= nodal accelerations, $\dot{u}$=nodal velocities and $f(t)$ is the difference between the external $f^{ext}(t)$ and internal nodal forces $f^{int}(t)$ according to Equation 3.

$$f(t) = f^{ext}(t) - f^{int}(t)$$

(3)

During the 3D dynamic analysis, the developing structural inertia forces are automatically taken into account. On the contrary, when running a static analysis, the nodal accelerations and velocities are equal to zero, which simplifies Equation 2 accordingly.

3.3 Results of static and rate dependent dynamic analysis

The static pull-out capacity was calculated using static analysis. During the static analysis, no rate effect and structural inertia forces were taken into account ($\ddot{u}=0$, $\dot{u}=0$). The static concrete cone capacity was $N_{u,stat,ref}=59,9$ kN with $h_{ef}=70$ mm and $N_{u,stat,ref}=41,4$ kN with $h_{ef}=50$ mm, respectively. These loads were considered as static reference loads, to which later the increased concrete cone capacity due to higher loading rates were compared. The headed studs were then subjected to 10, 100, 1000, 10.000 and 100.000 mm/s displacement rates (using displacement control) and the ultimate concrete cone capacity was calculated using the rate dependent dynamic analysis. During
the rate sensitive dynamic analysis, the rate dependent constitutive law as well as the structural inertia forces were taken into account. The relative concrete cone resistance as a function of the displacement rate are shown on Figure 3a.

Figure 3: Results of rate sensitive dynamic analysis: Effect of the displacement rate (a) and initial loading rate (b) on the ultimate concrete cone capacity

It was observed that the relative increase in the ultimate loads due to dynamic loading with \( h_{cf}=50 \text{ mm} \) are slightly above of those with \( h_{cf}=70 \text{ mm} \). This is explained by the smaller ultimate displacement of the \( h_{cf}=50 \text{ mm} \) headed stud. Subsequently, the same displacement rate [mm/s] applied on the \( h_{cf}=50 \text{ mm} \) headed stud resulted in higher loading rates [kN/s] than on the \( h_{cf}=70 \text{ mm} \) headed stud, thus the relative increment was also expected to be higher. Because of the displacement control, the loading rate was not linear, wherefore the distinction between secant and initial loading rate must be made during the evaluation according to Figure 4.

Figure 4: Distinction between secant loading rate (\( \dot{\varepsilon}_{\text{sec}} \)) and initial loading rate (\( \dot{\varepsilon}_{\text{ini}} \))

An example for a load-time relation is shown in Figure 4. The secant loading can be expressed as the peak load divided by the elapsed loading time. On the contrary, the initial loading rate only accounts for the initial constant part of the load-time curve. The end of the initial part was set to \( 0,5 \cdot N_0 \) during the determination of the initial loading rate. Subsequently, the initial loading rate is higher than (or min. equal to) the secant loading rate. The increment of the concrete cone capacity as a function of the initial loading rate is shown in Figure 3b. The two curves match very well, which let us conclude that the size effect is not significant, when comparing the results of the two investigated models (\( h_{cf}=50 \text{ mm} \) and \( h_{cf}=70 \text{ mm} \)). The size effect can however be significant, when having bigger differences in the embedment depths according to Ozbolt² (Figure 1a). The change of the failure
mode from concrete cone to local concrete failure of the fastener at very high displacement rate was observed (Figure 5), which is in good agreement with the previous findings of Ožbolt\(^2\).

![Image of failure modes at different displacement rates](image)

**Figure 5:** Failure mode at different displacement rates a) 10 mm/s, b) 1000 mm/s c) 100,000 mm/s

The next type of the executed numerical calculations consisted of 3 phases: (1) pre-loading with a defined displacement rate using rate dependent dynamic analysis, (2) unloading and (3) quasi-static pull-out of the headed stud using static analysis (no rate effect). The first and the second loading stages simulate the impact pre-loading (N<N\(_u\)) of the fastener, which is followed by an immediate spontaneous unloading. The residual capacity phase was carried out on the previously loaded (pre-damaged) model, while taking into account the pre-damage due to impact loading. The applied relative preload at a given displacement rate was expressed as a percentage of the ultimate load obtained for the corresponding displacement rate. For comparison, the loading, unloading, and following static pull-out was also made using rate-insensitive static analysis, where the applied relative preload was a percentage value of the reference static load. The load-displacement curves obtained from the numerical analysis are shown in Figure 6.

![Image of load-displacement curves](image)

**Figure 6:** Load-displacement curves of impact preloading (100 mm/s and 1000 mm/s), unloading and static pull-out of h\(_{cf}\)=70 mm headed stud

For a better graphic interpretation of the results, the relative residual capacities were plotted versus the relative applied preload (Figure 7). The calculated failure points were linearly regressed, and the corresponding trends including the slopes are also given in Figure 7. It can be seen that a loading cycle using static analysis leads only to a slight loss of the residual capacity, which is also in good agreement with the tests performed on headed studs under cyclic loads followed by pull out (Hoehler, 2006) in case of concrete cone failure. On the contrary, a significant degradation in the residual concrete cone capacity was observed, when the relative preload was applied with increased loading rate (rate sensitive dynamic analysis). The degradation is linear proportional with the applied load level as well as with the loading rate. Approximately 25% deflection of the concrete cone capacity was observed, when 70% relative initial preload with 1000 mm/s loading rate was applied.
The degradation is less, only 20% if the same initial load with only 100 mm/s loading rate was applied.

Figure 7: Degradation of the residual concrete cone capacity due to preloading (static preloading, 100mm/s preloading, 1000 mm/s preloading) of h_p=70 mm headed stud

4 Experimental verifications

4.1 Testing concept

The first aim of the experimental investigations was to verify the numerically predicted rate dependent ultimate concrete cone capacity above 40,000 kN/s loading rate. Furthermore, it was aimed to verify the detrimental effect of impact pre-loading on the concrete cone capacity. The impact preloads were applied on the fasteners using calibrated steel predetermined breaking points. The impact loading tests were carried out on an epoxy-based bonded anchor system and on Ø22 mm cast-in headed studs. The installation parameter i.e. anchor diameter and the embedment depth of the bonded system was selected in the way that other failure modes i.e. steel failure or pull-out (bond failure) were unlikely to occur. The effective anchorage depth was 50 mm in the case of both anchor systems. The executed tests included static reference tests, impact-loading tests to failure and impact preload followed by static pull-out tests. The protocol used in this combined test is similar to that used during the numerical analysis consisting of the pre-loading with high loading rate, followed by unloading and static pull-out tests for the assessment of the residual concrete cone capacity.

4.2 Tested materials

Concrete cylinders of diameter Ø320 mm and height h=150 mm served as base material for the impact loading tests. The used concrete mixture is shown in Table 2. The headed stud was fixed to the steel formwork prior to casting. The post-installation of the bonded anchors into the hardened concrete specimens was carried out according to the corresponding product approval.

Table 2: Concrete mixtures

<table>
<thead>
<tr>
<th>Batch No.</th>
<th>Aggregates</th>
<th>Cement CEM I 32.5 R</th>
<th>Water</th>
<th>w/c</th>
<th>Slump test /consistency class</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0-2</td>
<td>2-8</td>
<td>8-16</td>
<td>[kg/m³]</td>
<td>[kg/m³]</td>
</tr>
</tbody>
</table>
| 1         | 787 | 499 | 744         | 175   | 240     | 0,73    | 39 | F2
| 2         | 786 | 503 | 739         | 178   | 240     | 0,74    | 41 | F2
| 3         | 708 | 465 | 689         | 185   | 265     | 0,75    | 43 | F3

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4.3 Test arrangement

The available experiments show that high loading rates (>50,000 kN/s) cannot be exceeded by using conventional servo-hydraulic testing cylinders\textsuperscript{5,6}. To provide higher loading rates, a new test apparatus was designed (Figure 8), which uses the potential energy of the impactor to generate the required kinetic energy and hence the velocity at the time of impact. The actual velocity of the falling object can be expressed as a function of the height of fall of the impactor and the gravitational acceleration. The impact of the weight discs (which were connected to each other) at the top steel plate moved the rigid steel frame rapidly downwards, which was connected to the fastener through a spherical calotte. The load was measured directly below the nut using compact piezo load washer (Kistler 9061A). The vertical displacement of the fastener was measured indirectly using piezo accelerometer (Kistler 8005). Although the anchor displacement was evaluated by the double integration of the acceleration, the evaluation of the displacement data is not the scope of this recent paper and the comparison of the experimental results with the numerically obtained results was made using on the initial loading rate versus the loads. The data was recorded with 100 MHz measurement frequency.

![Figure 8 a) Schematic of the test arrangement b) test arrangement c) concrete specimens](image)

4.4 Impact tests to failure

According to the testing concept, impact-loading tests until failure were carried out to investigate the ultimate concrete cone resistance. During all executed impact-loading tests, the same weight (160 kg) and free-fall height (1 m) was used. Despite the different load-transfer mechanism (mechanical interlock, bond) of headed studs and bonded anchors, all tests failed by full-size concrete cone under tension, which enabled the realistic quantitative comparison of the results. The comparison of the experiments and the numerically obtained results is shown in Figure 9. For the reasonable comparison, the initial loading rate was calculated for all executed impact-loading tests and the applied displacement rates in the numerical analysis was also calculated into initial loading rates. The new test results as well as the results of Tóth et. al. 2016\textsuperscript{7} are in a very good agreement with the
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numerically obtained relative concrete cone resistances. Comparing to the measured static reference load (45 kN), the increment of 91% (86 kN) of the concrete cone capacity was achieved at ca. 90.000 kN/s loading rate. The obtained experimental results on the rate dependent concrete cone behavior verify the numerically obtained results with great accuracy and to the knowledge of the author, these are the first results in the fastening technology exceeding 50.000 kN/s loading rate.

Figure 9: Influence of loading rate on the concrete cone capacity: comparison of the new experimental results, and the experimental results from Tóth et. al. 2016 with numerical results

4.5 Impact pre-loading tests

During the impact pre-loading, the pre-loads were applied on the fasteners using predetermined breaking points. Commercial threaded rods of size M8 with different strength classes (8.8, 10.9, 12.9) served as breaking points. They were built in the force flux as it is schematically shown in Figure 8a. The ultimate load of the predetermined breaking point corresponded the load, with which the fastener was preloaded. Prior to testing, preliminary impact-loading “calibration” tests were carried out on the used steel breaking points to get information about their rate dependent behavior and to being able to select such threaded rods, whose ultimate resistance is smaller than the concrete cone capacity at the corresponding loading rate. The progress of the executed tests consisted of 3 steps: (1) impact preloading with the new test apparatus using threaded rods as predetermined breaking points, (2) spontaneous unloading after failure of the predetermined breaking point and finally, (3) a static unconfined pull-out test after removing the preloaded concrete specimen from the test apparatus. The impact preloading tests were carried out with 1 m, free-fall height and 160 kg weight. Due to the scatter in the ultimate resistance of the breaking points, the relative preload was different in all executed tests. After each preloading, the actual initial loading rate was calculated. This was required for the determination of the relative preload. At the calculated loading rate of preload, the ultimate concrete cone resistance was read off from Figure 9. This corresponds theoretically the load, under which the anchor would have failed by concrete cone at that specific loading rate, so the relative preload expresses the ratio of the measured preload and the ultimate concrete cone capacity at the specific loading rate. The results of the preloading tests including the residual concrete cone capacities are shown in Table 3 and Table 4. Similar to the interpretation of numerical results, the test results were plot in relative residual capacity-relative applied preload graphs (Figure 10). The experiments confirm the detrimental effect of the preload applied at high loading rates.
Table 3: Results of preloading tests and residual capacity (Headed studs, $h_{ef}=50$ mm)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$N_{preload}$ [kN]</th>
<th>Loading rate of $N_{preload}$ [kN]</th>
<th>$N_{u,imp,ref,ini}$ (read-off) [kN]</th>
<th>$N_{preload}/N_{u,imp,ref}$</th>
<th>$N_{u,stat,ref}$ [kN]</th>
<th>$N_{u,res,stat}$ [kN]</th>
<th>$N_{u,res,stat}/N_{u,stat,ref}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>66.55</td>
<td>68.600</td>
<td>68.9</td>
<td>0.96</td>
<td>45.03</td>
<td>40.1</td>
<td>0.89</td>
</tr>
<tr>
<td>2</td>
<td>63.74</td>
<td>70.830</td>
<td>69.8</td>
<td>0.91</td>
<td>45.03</td>
<td>40.9</td>
<td>0.90</td>
</tr>
<tr>
<td>3</td>
<td>49.57</td>
<td>55.000</td>
<td>59.9</td>
<td>0.83</td>
<td>45.03</td>
<td>37.92</td>
<td>0.84</td>
</tr>
<tr>
<td>4</td>
<td>65.48</td>
<td>82.900</td>
<td>72.9</td>
<td>0.89</td>
<td>45.03</td>
<td>44.37</td>
<td>0.98</td>
</tr>
<tr>
<td>5</td>
<td>61.21</td>
<td>67.900</td>
<td>66.6</td>
<td>0.92</td>
<td>45.03</td>
<td>37.96</td>
<td>0.84</td>
</tr>
<tr>
<td>6</td>
<td>51.71</td>
<td>41.200</td>
<td>57.6</td>
<td>0.89</td>
<td>45.03</td>
<td>42.72</td>
<td>0.95</td>
</tr>
</tbody>
</table>

Table 4: Results of preloading tests and residual capacity (Bonded anchor, $h_{ef}=50$ mm)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>$N_{preload}$ [kN]</th>
<th>Loading rate of $N_{preload}$ [kN]</th>
<th>$N_{u,imp,ref,ini}$ (read-off) [kN]</th>
<th>$N_{preload}/N_{u,imp,ref}$</th>
<th>$N_{u,stat,ref}$ [kN]</th>
<th>$N_{u,res,stat}$ [kN]</th>
<th>$N_{u,res,stat}/N_{u,stat,ref}$</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>53.18</td>
<td>133.000</td>
<td>116.9</td>
<td>0.45</td>
<td>41.03</td>
<td>40.87</td>
<td>0.99</td>
</tr>
<tr>
<td>2</td>
<td>44.76</td>
<td>41.800</td>
<td>53.3</td>
<td>0.84</td>
<td>41.03</td>
<td>39.60</td>
<td>0.96</td>
</tr>
<tr>
<td>3</td>
<td>63.07</td>
<td>73.000</td>
<td>63.2</td>
<td>0.99</td>
<td>41.03</td>
<td>36.96</td>
<td>0.90</td>
</tr>
<tr>
<td>4</td>
<td>59.06</td>
<td>64.200</td>
<td>60.7</td>
<td>0.97</td>
<td>41.03</td>
<td>40.87</td>
<td>0.99</td>
</tr>
<tr>
<td>5</td>
<td>50.91</td>
<td>53.000</td>
<td>53.3</td>
<td>0.95</td>
<td>41.03</td>
<td>38.7</td>
<td>0.94</td>
</tr>
<tr>
<td>6</td>
<td>50.64</td>
<td>52.500</td>
<td>54.9</td>
<td>0.92</td>
<td>41.03</td>
<td>34.73</td>
<td>0.84</td>
</tr>
<tr>
<td>7</td>
<td>35.04</td>
<td>49.700</td>
<td>46.25</td>
<td>0.76</td>
<td>35.04</td>
<td>28.48</td>
<td>0.81</td>
</tr>
</tbody>
</table>

Figure 10: Residual concrete cone capacity versus relative applied preload a) Ø22 $h_{ef}=50$ mm headed stud, b) M16 $h_{ef}=50$ mm bonded anchor

5 Verification

5.1 Verification concept

The numerical analysis was only carried out with certain loading rates. The calculated initial loading rates of the degradation lines in Figure 7 are 0.1 kN/s, 59.500 kN/s and 620.000 kN/s, respectively. Subsequently, the direct comparison of the experimental data with the numerically obtained degradation lines was not possible because of the different initial loading rates of the applied preload in each executed impact-loading test. For the comparison, further degradation lines were determined. It was also observed that the degradation lines from the numerical analysis are straight lines (Figure 7). Regarding these findings, the determination of further degradation lines between the existing
degradation lines in Figure 7 can be made assuming either linear or logarithmic distribution. Since all calculated loading rates of the applied preloads are in the range of 40-130 MN/s, which is exactly the intermediate phase between the two effects, the determination of new degradation lines between the existing numerically obtained degradation lines can be made assuming linear or logarithmic distribution in. As it is shown in Figure 7, one linear degradation line reads (Eq.4):

\[ y = 1 - m \cdot x \] (4)

Where \(m\) is the slope of the degradation curve, \(x\) is the relative applied preload and \(y\) is the relative residual concrete capacity. When assuming linear distribution of further degradation lines between the existing degradation lines (1 kN/s, 59.500 kN/s, 620.000 kN/s), the slope \(m\) of the degradation curve in function of an arbitrary chosen loading rate (\(\kappa\)) reads (Equation 5, Equation 6):

If \(\kappa < 59.500\) kN/s:

\[ m = -0.0485 - \left[ (0.2263 - 0.0485) \left( \frac{\kappa}{59500} \right) \right] \] (5)

If \(59.500 < \kappa < 620.000\) kN/s:

\[ m = -0.2263 - \left[ (0.322 - 0.2263) \left( \frac{\kappa - 59500}{620000 - 59500} \right) \right] \] (6)

When assuming logarithmic distribution of the new degradation lines between the existing degradation lines, the slope \(m\) of the degradation curve in function of an arbitrary chosen loading rate (\(\kappa\)) reads (Equation 7, Equation 8):

If \(5950 < \kappa < 59.500\) kN/s:

\[ m = -0.0485 - \left[ (0.2263 - 0.0485) \log_{10} \left( 1 + \frac{\kappa}{59500} \right) \right] \] * 10 \] (7)

If \(59.500 < \kappa < 620.000\) kN/s:

\[ m = -0.2263 - \left[ (0.322 - 0.2263) \log_{10} \left( \frac{\kappa}{620000} \right) \right] \] * 10 \] (8)

With this presented concept, a degradation line to each individual test point contained in Figure 10a and Figure 10b could be determined. The verification was carried out for each executed test separately and the numerically predicted relative residual capacity was compared to the measured relative residual capacities. An average deviation of 8% (linear) and 11% (logarithmic) between the numerical and experimental results was calculated.

6 Conclusions and Outlook

In this study, the effect of impact preloading on the residual concrete cone capacity of fasteners was investigated numerically and experimentally. The numerical analysis showed the linear relation between the applied load level and the loss of the concrete cone capacity. Furthermore, the numerical results showed that the loading rate of the applied preload has a strong influence on the degradation as well. The results were interpreted with degradation lines. For the experimental verification, a new test setup was designed and built, with which loading rates in the range of 40-130 MN/s could be induced and the impact preload was applied using predetermined breaking points. With the executed impact-loading tests the rate dependent Microplane model was verified (up to 130 MN/s). The numerically predicted detrimental effect of the impact preload on the residual concrete cone capacity was also verified. The calculated deviation between the numerical analysis and the test results was 8% to 10%, which can be presumably further decreased by more impact-loading and impact-preloading tests. The numerically obtained results predict more pronounced damage due to impact-preloading. Consequently, the M2-O Microplane Model provides robust and conservative results.
Based on the first available verifications on the concrete cone failure of fasteners, the rate dependent Microplane Model can presumably capture the concrete behavior under impact loads and under impact preloads in general, if tension failure of concrete is expected.

7 Acknowledgement

The Authors are grateful to Prof. Ozbolt for the discussions and support received during the numerical investigations. The Authors greatly acknowledge the technical support of Mrs. Katrin Allmendinger (MPA Stuttgart) during the execution of the impact-loading tests.

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EFFECT OF CRACK OPENING ON THE BEHAVIOUR OF MONO-STUD PLATES UNDER SEISMIC CONDITIONS

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ABSTRACT

Anchor plates are commonly used in nuclear power plants as connection between equipment and reinforced concrete structures. For building of importance class IV, prEN 1992-4 recommends to use fasteners of seismic performance Category C2, however there are no specific codes for the assessment of anchoring plates under seismic action. An experimental campaign was carried out in order to investigate the behaviour of anchor plates by adapting the European protocols for the seismic assessment of post-installed anchors. Anchor plates usually have at least four fasteners, thus complicating the installation in presence of high density of reinforcement. New innovative solutions with only one headed anchor are investigated: a more classic solution made with a single stud welded in the centre of the steel plate and a more innovative one made with a piece of an IPE profile, welded in the centre of the steel plate. Varying crack width tests with constant tensile load were performed. Both centred and eccentric tension were applied during the tests in order to simulate the installation of an eccentric welded support. Capacity reduction after cyclic tests is evaluated and compared with CCD method. The behaviour under eccentric tension is investigated in order to define the transfer mechanism of the bending moment to the concrete.

1 Introduction

Anchoring plates are a common solution to create a link between equipment and reinforced concrete structures in nuclear power plants. Despite the wide diffusion of post-installed fasteners in recent years, many constructive details require cast-in place anchors in specific situations of high density of reinforcement that lead to complex installation procedures.

Anchoring plates are composed by headed anchors welded to the steel plate. Some typical solutions are realized by using four studs1,2, but many issues for installation in regions with high reinforcement ratio are expected. Electricité de France (EDF) developed innovative solutions for anchor plates to be used in these design conditions under seismic loads, also in presence of an eccentric load with respect to the plate centroid.

Two experimental campaigns were carried out by Department of Civil and Environmental Engineering of Politecnico di Milano (DICA) for the assessment of these anchoring solutions: the first one covered the tensile behaviour, while the second one analysed the shear behaviour. Due to the lack of specific protocols for the assessment of cast-in place anchors, European protocols were
adapted, in particular ETAG001-Annex E that describes procedures for post-installed anchor qualification. The seismic performance of fasteners subjected to seismic loading is represented by performance categories C1 and C2. Performance category C1 provides fastener capacities only in terms of resistances at ultimate limit state, while performance category C2 provides fastener capacities in terms of both resistances at ultimate limit state and displacements at damage limitation state and ultimate limit state. The requirements for category C2 are more stringent compared to those for category C1. The performance category valid for a fastener is given in the corresponding European Technical Product Specification. Table 1 relates the seismic performance categories C1 and C2 to the seismicity level and building importance class. The level of seismicity is defined as a function of the product $a_g \times S$, where $a_g$ is the design ground acceleration on Type A ground and $S$ the soil factor both in accordance with EN 1998-1.

Table 1: Recommended seismic performance categories for fasteners

<table>
<thead>
<tr>
<th>Seismicity level$^a$</th>
<th>Importance Class acc. to EN 1998-1:2004, 4.2.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Class</td>
<td>I</td>
</tr>
<tr>
<td>Very low$^b$</td>
<td>$a_g \times S \leq 0.05 \text{ g}$</td>
</tr>
<tr>
<td>Low</td>
<td>$0.05 \text{ g} &lt; a_g \times S \leq 0.1 \text{ g}$</td>
</tr>
<tr>
<td>&gt; low</td>
<td>$a_g \times S &gt; 0.1 \text{ g}$</td>
</tr>
</tbody>
</table>

$^a$ The values defining the seismicity levels are subject to a National Annex. The recommended valued are given here.$^b$

$^c$ $a_g$ = design ground acceleration on type A ground (EN 1998-1:2004, 3.2.1).

$^d$ $S$ = soil factor (EN 1998-1:2004, 3.2.2)

$^e$ C1 for fixing non-structural elements to structures (Type 'B' connections)

$^f$ C2 for fixing structural elements to structures (Type 'A' connections)

In this paper, results of crack movement tests on two anchors plates according to the C2 category are presented. Tests were carried out at Laboratorio Prove Materiali, Strutture e Costruzioni (LPMSC) of Politecnico di Milano. Two types of loads were applied, related to point of application: centred axial load and eccentric load with respect to the plate’s vertical axis. Results of tests are discussed to evaluate contributions of each component (stud, plate, IPE profile and concrete). Comparison between results and theoretical predictions of CCD method are reported to prove its applicability as design method for these innovative solutions.

2 Experimental investigation

2.1 Mono-stud anchor plates

Two anchoring plates were tested:
- Solution S1 (Figure 1): a more classic solution made with a stud of 22 mm diameter and 200 mm height welded in the centre;
- Solution S4 (Figure 1): a more innovative solution made with a piece of IPE profile of steel S235 of 73 mm length welded in the centre.

Two other solutions, named S2 and S3, were also developed but they are not examined in this paper. Class S355 steel plates are intended to accommodate a welded support for equipment with a maximum distance of 50 mm with respect to the centroid axis. Table 2 summarizes geometrical details of mono-stud plates.

![Figure 1: Mono-stud anchor plates: Solution S1 (left) and Solution S4 (right).](image)

According to prEN 1992-4, the embedment depth is 212 mm and 155 mm for solution S1 and solution S4, respectively.

<table>
<thead>
<tr>
<th>Solution</th>
<th>Geometric Details</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Plate (mm)</td>
</tr>
<tr>
<td>S1</td>
<td>200×200×22</td>
</tr>
<tr>
<td>S4</td>
<td>200×200×22</td>
</tr>
</tbody>
</table>

### 2.2 Concrete slabs

A reinforced concrete slab of 500 mm width, 340 mm depth and 1440 mm length with two anchoring plates was selected for crack movement tests. The installation of two samples for each slab is required for the complete development of concrete cone. This target is achieved by setting spacing of cracks inducers at a distance of 630 mm in formworks used to cast the specimens. By operating in this direction, the cone superimposition was avoided.

Slabs were casted with class C20/25 concrete and reinforced with M20 steel class B7 threaded rods. For each slab, at least three compressive tests on cubes of 150 mm side were performed to measure the average compressive strength of the concrete. In Figure 2, a sample before testing is shown.
2.3 Protocols adopted

Nowadays there is a lack of information about the assessment of cast-in place anchors under seismic action. On the other hand, post-installed anchors are qualified according to specific procedures (for a review, see Mahrenholtz\textsuperscript{5}), which are provided by ETAG001 Annex E\textsuperscript{6} and ACI 355.2\textsuperscript{7} in Europe and United States, respectively. The complete experimental campaign is composed by the following tests:

- Monotonic reference tension test;
- Pulsating tension test with a fixed value of crack width;
- Monotonic reference shear test;
- Alternating shear test with a fixed value of crack width;
- Tension load with varying crack width.

The protocol of the last test series was developed by performing rainflow counting of the curvature histories extracted from nonlinear history analyses of a suite of building models\textsuperscript{8}. Protocols of ETAG001 Annex E\textsuperscript{6} were adapted for the scope of this work. The present paper presents and discusses test results for monotonic reference tension tests (identified as C2.1) and varying crack width tests (identified as C2.5) only.

2.4 Load

According to EDF, anchoring plates are designed to allow the installation of supports for equipment with a maximum eccentricity of 50 mm. Consequently, two limit cases are identified:

- Centred shear with the load direction coincident with the axis of the anchor;
- Eccentric shear with 50 mm of eccentricity with respect to the axis of the anchor.

For the second case the eccentricity was set on the same side in which crack develops respect to crack inducers. The procedure for tests with varying crack width (C2.5 test series) is consistent with ETAG001-Annex E\textsuperscript{6}. After the installation, the load $N_{w1}$ was applied to the anchor plate and a crack cycling protocol according to Table 3 with a cycling frequency equal to 0,5 Hz is started. The load is successively increased to the value $N_{w2}$, as reported in Table 3.

In each cycle, the crack was closed by applying a centric compression force $C_{\text{test}}$ to the slab that is equal to:

$$C_{\text{test}} = 0,1 * f_{c,C2.5} * A_g$$  \hspace{1cm} (1)

with
\[ A_g = \text{cross section area of the test member} \ [\text{mm}^2] = b \times h, \text{ where } b \text{ and } h \text{ are width and thickness of the test member, respectively; } \]

\[ f_{c,C2.5} = \text{mean compressive strength of concrete measured on cubes at the time of testing of the test series C2.5} \ [\text{N/mm}^2]. \]

\[ N_{w1} \text{ and } N_{w2} \text{ are determined as follows:} \]

\[ N_{w1} = 0.4 \times N_{u,m,C2.1a} \times \left( \frac{f_{c,C2.5}}{f_{c,C2.1a}} \right)^{0.5} \]

\[ N_{w2} = 0.5 \times N_{u,m,C2.1a} \times \left( \frac{f_{c,C2.5}}{f_{c,C2.1a}} \right)^{0.5} \]

Table 3: Number of cycles and applied load for different crack widths values - C2.5 series

<table>
<thead>
<tr>
<th>Crack width ( \Delta w ) [mm]</th>
<th>Number of cycles</th>
<th>Anchor load</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>20</td>
<td>( N_{w1} )</td>
</tr>
<tr>
<td>0.2</td>
<td>10</td>
<td>( N_{w1} )</td>
</tr>
<tr>
<td>0.3</td>
<td>5</td>
<td>( N_{w1} )</td>
</tr>
<tr>
<td>0.4</td>
<td>5</td>
<td>( N_{w1} )</td>
</tr>
<tr>
<td>0.5</td>
<td>5</td>
<td>( N_{w1} )</td>
</tr>
<tr>
<td>0.6</td>
<td>5</td>
<td>( N_{w2} )</td>
</tr>
<tr>
<td>0.7</td>
<td>5</td>
<td>( N_{w2} )</td>
</tr>
<tr>
<td>0.8</td>
<td>4</td>
<td>( N_{w2} )</td>
</tr>
</tbody>
</table>

The residual capacity test was carried out with a crack width equal to \( \Delta w = 0.8 \) mm. The complete test program is reported in Table 4.

Table 4: Test program.

<table>
<thead>
<tr>
<th>Test code</th>
<th>Solution</th>
<th>Concrete</th>
<th>Crack width (mm)</th>
<th>Eccentricity (mm)</th>
<th>N. of tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1-C21a-C</td>
<td>S1</td>
<td>C20/25</td>
<td>0.8</td>
<td>-</td>
<td>6</td>
</tr>
<tr>
<td>S1-C21a-E</td>
<td>S1</td>
<td>C20/25</td>
<td>0.8</td>
<td>50</td>
<td>6</td>
</tr>
<tr>
<td>S4-C21a-C</td>
<td>S4</td>
<td>C20/25</td>
<td>0.8</td>
<td>-</td>
<td>7</td>
</tr>
<tr>
<td>S4-C21a-E</td>
<td>S4</td>
<td>C20/25</td>
<td>0.8</td>
<td>50</td>
<td>5</td>
</tr>
<tr>
<td>S1-C2.5-C</td>
<td>S1</td>
<td>C20/25</td>
<td>0.1÷0.8</td>
<td>-</td>
<td>8</td>
</tr>
<tr>
<td>S1-C2.5-E</td>
<td>S1</td>
<td>C20/25</td>
<td>0.1÷0.8</td>
<td>50</td>
<td>9</td>
</tr>
<tr>
<td>S4-C2.5-C</td>
<td>S4</td>
<td>C20/25</td>
<td>0.1÷0.8</td>
<td>-</td>
<td>5</td>
</tr>
<tr>
<td>S4-C2.5-E</td>
<td>S4</td>
<td>C20/25</td>
<td>0.1÷0.8</td>
<td>50</td>
<td>5</td>
</tr>
</tbody>
</table>
2.5 Test setup and execution

A special apparatus composed of two reaction frames was used for tests. The first frame lays horizontally and it is fixed to the strong floor (Figure 3). It is equipped with a hydraulic jack that is able to cyclically open and close (including compressing) the crack; the maximum load capacity is 1000kN. The second reaction frame is movable and it was positioned on the concrete slab. It has a load capacity of 300kN and it was used to apply the tension load to anchoring plates.

Eight linear displacement transducers of different length (LVDTs) were installed on concrete slabs to measure both crack width and displacements of anchor plates. LVDTs layout on the slab was according to ETAG001 Annex E (EOTA, 2013) monitoring the crack width both at the surface and on the slab sides at a distance from the surface equal to the anchor embedment depth.

Linear displacement transducers (LVDTs) of different length were installed on concrete slabs to measure both crack width and displacements of anchor plates. A special sleeve to connect steel frame and anchor plate was designed, with two different positions horizontally spaced of 50 mm, such to carry out centred and eccentric tests.

![Figure 3: Test apparatus for crack opening](image)

3 Test results

Test results are summarized in Table 5, where $f_{c,\text{test}}$ is the compressive strength of the concrete batch, C indicates concrete cone failure, BO blow-out failure, $\delta_{0.5Nu}$ is the displacement at the 50% of the peak load, $N_{u,m,\text{test}}$ and $N_{u,m}$ are the average normalized tensile load. The normalization of the ultimate tensile load is calculated using the following equation:

$$N_{u,m}(f_c) = N_{u,m,\text{test}} \cdot \left(\frac{f_c}{f_{c,\text{test}}}\right)^{0.5}$$

(4)
It can be noticed how crack cycling had no effect on the anchor capacity for both anchor types. The corresponding load-displacements curves for all C2.5 residual tests are reported in Figure 6 and 7. Stiffness of Solution 4 is higher than Solution 1, due to a different mechanism of load transfer between IPE profile and concrete. In fact, the not conventional dimensions (if compared to a typical anchor) of the base flange reduces the concrete damage in compression; on the other hand, the ultimate capacity for Solution 1 is 40% higher than for Solution 4 due to the larger embedment depth. Figure 8 and Figure 9 report two typical concrete cone failures for Solution 1 and 4, respectively. Additionally, as shown also by numerical analysis, the crack propagation angle, measured from the loading direction, increases with increase of head size.

Figure 6: C2.5 tests, load/displacement curves for Solution 1, centred (left) and eccentric (right)
Discussion of results

4.1 Behaviour under crack movement

None of the samples failed during crack movement tests, so all the eccentric and centred tests were conducted without any load reduction in the constant load portion of the test. For both Solution 1 and 4 ultimate capacity is not affected by crack cycling. This is probably due to a negligible influence of crack cycling on the load transfer mechanism under the head of the stud or the IPE flanges.

Solution 4 generally has lower displacements than Solution 1. Figure 10 reports the rotational capacity developed by both solutions in eccentric tests accounting (Figure 10 – left) or neglecting (Figure 10 – right) the displacement cumulated during the crack cycling (where relevant).

The rotational capacity is defined as the ratio between the difference in the vertical displacements of the fixture measured in correspondence of the T-transducers (on the side in tension) and C-transducers (on the side in compression) - calculated as \[ \delta_{T1-T2}(0.5\,\text{Num},i)\] \[\delta_{C1-C2}(0.5\,\text{Num},i)\] - over the distance between the transducers (equal to 210 mm). The rotation \(\theta_{\text{sls}}\) for a serviceability load level is calculated in correspondence of the 50% of the mean failure load \(L_{\text{f}}\) (for cyclic tests the value takes into account also the displacement accumulated during cycling phase).
Solution 1 develops a high global rotational capacity at the failure under both monotonic loading and -globally- including crack cycling. Solutions 4 is definitively stiffer even though the irreversible displacement cumulated during crack cycling is significantly lower with respect to Solution 1.

4.2 Comparison with predictive models

Under eccentric loading, the following load transfer mechanism is supposed to activate. It is assumed that the bending moment due to eccentricity is totally transferred to the base material by contact with the concrete around the anchor, as reported in Figure 11. Consequently, the tensile force transferred to the fastener does not depend on the value of eccentricity, as confirmed by test results.

Figure 11: Transfer mechanism of stresses

Figure 13 compares test results with the theoretical predictions of prEN 1992-43 (based on the Concrete Capacity Design – CCD10) as a function of the concrete compressive strength. The load capacity in tension is expressed by the following equations:

\[ N_{Rm,c}^0 = k_1 \times \sqrt{\frac{f_c}{h^{1.5}}} \]  
\[ N_{Rm,cb}^0 = k_s \times \sqrt{A_h \times \sqrt{f_c}} \]  
\[ N_{Rm,s}^0 = A_s \times f_u \]

for concrete cone, concrete blow-out and steel failure modes, respectively. According to Eligehausen et al.10, the coefficients \( k_1 = 11.5 \) and \( k_s = 11.2 \) are adopted, with reference to the mean capacity. It can be noticed how the capacity is generally well predicted by the CCD approach for Solution 1, even though slightly overestimated.
For Solution 1 it was observed that test results are strictly dependent from the installation of studs, in particular a correct welding is achieved when the stud detects an angle of 90° with the anchor plate. Markers highlighted in red are related to not-vertical configuration of the studs, as shown in Figure 13. The displacement $\delta$ indicates the deviation of stud from the correct perpendicular position.

![Figure 12: Geometrical deviation of stud welded on plate](image)

![Figure 13: Comparison between tested and theoretical capacity for Solution 1 and 4 as function of concrete compressive strength](image)

On the contrary, the CCD approach significantly overestimates the tensile capacity for Solution 4, probably due to the lower contact pressures between the concrete and the embedded flange of the IPE profile, which is wider than the typical heads of the studs investigated in the past and which represent the basis for the CCD approach. Consequently, an alternative assessment may consist in defining different values for the $k_1$ coefficient to be used in Equation 5. The best fit of experimental data (reported in Figure 14) returns a value of $k_1$ coefficient equal to 10.0 and 8.0 for the average capacity of Solution 1 and Solution 4, respectively.
Figure 14: Comparison between tested and theoretical capacity (best fit) for Solution 1 and 4 as a function of concrete compressive strength

5 Conclusions

In this paper, an experimental campaign on seismic behaviour of two different mono-stud plates is presented, one incorporating a single stud (Solution 1), the other incorporating an IPE profile (Solution 4).

The investigated solutions differ in terms of both stiffness and capacity. In particulars Solution 1, due to its larger embedment depth (212 mm), allows to develop a higher capacity; on the other side Solution 4 has a higher stiffness due to unusual dimensions of base flanges that creates a good interlock in the concrete. For both solutions, the capacity is not affected neither by the crack cycling nor by the eccentricity of the applied load.

Test results were compared with prevision of CCD method\(^3\), which may be applied with a minor calibration of coefficients. This is particularly true for the plate incorporating an IPE profile (Solution 4), where the lower contact pressure in the load transfer zone require some adaptations of the existing design methods.

6 Acknowledgement

This research work was supported by EDF SEPTEN Company, Civil Engineering, Villeurbanne. Mr. A. El Yazidi and Mr. T. Roure at EDF are warmly thanked. The authors also thank all the members of technical staff of LPMSC, sector Structural Anchors, with a special regard to Mr. M. Dezio.

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EVALUATION AND DESIGN OF POST INSTALLED REBAR UNDER SEISMIC ACTIONS

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ABSTRACT

The bonding capacity of polymer adhesives has encouraged their application for steel bar anchors into concrete for structural purposes. Such application can be connection between walls and floors or columns to foundations. However, depending on seismic classification of the location where these anchorages are installed there may be a seismic resistance requirement. Since no European assessment document is applicable up to now, the French expert group of the “Commission Chargée de Formuler des Avis Techniques” (CCFAT) has established rules to be followed in order to get a qualification for seismic design of the post installed rebars (PIRs). These rules have been established on the basis of seismic tension tests performed on PIRs embedded in concrete members. This paper gives the pre-requisite evaluation of the mortar and the tests to be performed in order to get a seismic qualification. The assessment of the test results is explained for the seismic category C1 and C2 as defined in EOTA Technical Report 049 as well as the requirements during the installation procedure of the PIRs. The design of PIR is then covered. Finally an example of design and calculation is developed in order to show the scope of application of such assessment.

1 Introduction

Post-installed rebars is a structural joining technique allowing the connection and the load transfer between structural elements using steel rebars and adhesive polymers. Post-installed rebars were initially used in concrete constructions in retrofitting, extension and in repairing structures by adding new concrete sections to existing elements1. Over the time, the improvement in mechanical and adhesion properties of polymer adhesives have allowed to enhance the mechanical behavior of Post-installed rebars and led to achieve equivalent or even higher mechanical responses than cast-in place rebars at normal operating temperatures2,3. Thus, post-installed rebars have gradually substitute cast-in place rebars in new constructions for some applications by offering advantageous solutions and flexibility allowing to meet the high architectural requirements4. However, the mechanical behavior of post-installed rebars is directly linked to the mechanical properties of polymer resins, and it is necessary to ensure the polymer resin is suitable to work in cracked concrete which is to happen in case of a seismic event. Consequently, seismic hazard must be considered when designing post-installed rebars. Very few regulations exist today proposing methods to assess and to design the seismic resistance of chemically-bonded post-installed rebars. The Acceptance criteria AC3085 is one of these documents and the present publication is showing the approach used by the French
2 Post installed rebar system

2.1 The different post installed rebar systems

The use of injection systems is very widespread for the installation of PIRs and different types of PIRs coexist on the market. Their characteristics is given in the following paragraphs and their specificities are underlined. Nevertheless for all the methods described the drilling and cleaning steps of the installation procedure can be identical. First a bore hole is done in the concrete using an electric or pneumatic drilling machine or using a diamond core drilling machine. Then hole is cleaned in order to remove any drilling residue of the inside of the hole. Then the type of PIRs system can be chosen as one of the following.

2.1.1 Injection systems

After drilling and cleaning the hole, then a bi-component pre portioned resin most often of the Epoxy or Vinylester type is injected into the borehole with the help of a dispenser. The rebar is then introduced into the hole at the desired embedment depth and let polymerizes for few minutes to several hours depending on the type of polymer resin used. After curing the PIR is then connected to the concrete and its performances are considered as similar as to a cast in rebar.

2.1.2 Grouted systems

Grout systems are defined by “a mixture of cementitious material and water, with or without aggregate, proportioned to produce a pourable consistency without segregation of the constituents”. The grout has to be provided in a pourable consistency that means that the grout has to be mixed with a certain amount of water in order to be injected in the bore hole.

Because it cannot be ensure after setting that this amount of water has been precisely measured in order to meet the manufacturer specification, this kind of injection system has not been covered by the current seismic assessment.

2.1.3 Capsule systems

The capsule systems can be sometime used for the installation of PIRs. The way the installation is done for such system is as follow:

After drilling and cleaning the hole then one (several) bi-component pre portioned glass capsule (s) of resin is (are) dropped into the borehole. The rebar is then introduced into the hole by hand hammering that brakes the glass capsule. The two components of the polymer resin can then react together the polymerization of the rein can occurs. After curing the PIR is then connected to the concrete and its performances are considered as similar as to a cast in rebar.

2.2 Type of connection

Post-installed rebars are used for a large range of construction applications including connection of column or a wall to a foundation, end connection of slabs or beams to walls. These types of
applications are critical because they shall fulfil the Basic Work Requirements for mechanical resistance and stability as well as the safety in use of the Construction Product Regulation\textsuperscript{6}. Also failure of anchorages made with these products must be prevented as they would compromise the stability of the works, cause risk to human life and/or lead to considerable economic consequences. Examples are given below showing the requirements for several structural connections with indication of the embedment length of the PIR.

![Figure 1: Overlap joint for rebar connections of slabs and beams.](image1)

![Figure 3: End anchoring of slabs or beams, designed as simply supported.](image3)

![Figure 2: Overlap joint at a foundation of a column or wall where the rebar is stressed in tension.](image2)

![Figure 4: Rebar connection for components stressed primarily in compression; rebar is stressed in compression](image4)

2.3 Setting of post installed rebars

2.3.1 Manufacturer Published Installation Instructions

The PIRs will be subjected to a high level of load during a seismic event, consequently this is of a very high importance to follow the Manufacturer Published Installation Instructions (M.P.I.I.) of the manufacturer. The installation instructions contain many parameters that need to be followed in order that the performances published in the technical specifications can be achieved. These parameters include:
• The required drilling diameter that need to be fulfilled as it ensures to have the proper layer of resin all around the rebar,
• The drilling depth of the hole which allow to have the correct embedment depth of the rebar,
• The cleaning method recommended by the manufacturer. Additional recommendations are given in 2.3.2,
• Volume of resin to place into the hole in order to have the full depth of the bore hole filled with resin when the rebar is installed,
• Minimum and maximum installation temperature at which the bonded material can be used,
• Maximum gel time indicating how long the rebar can be placed into the resin before polymerization starts.
• Minimum curing time indicating how long one has to wait before the rebar can be loaded.

2.3.2 Cleaning the hole
The cleaning of the hole is one of the main step of the Manufacturer Published Installation Instructions where the installer need s to take a lot of care. Indeed the final bond strength of the connection can highly dependent of the cleaning efficiency of the hole. For that reason it is crucial to use the cleaning equipment provided by the manufacturer of the resin and described in the technical specification like European Technical Assessment which gives the diameter of the brush that need to be used, but also the kind of wire of the brush (e.g. steel or nylon) which has an impact on the final cleaning.

Figure 5: Examples of brush with nylon or steel wire.

In addition the dust and debris that stay in the hole after drilling need to be removed. Indeed the debris that will stay at the bottom of the hole will reduce the embedment depth if they stay in the bottom during injection of the resin. Also the dust that cover the wall of the hole after drilling will create a thin layer that will prevent the resin to bond to the concrete and thus can reduce the bond strength of the connection. For these reasons it is of importance to clean the hole using the correct sequence of blowing and brushing operations required by the M.P.I.I.

3 Methodology

3.1 Objectives
For use in seismic zones, the PIRs must be designed by checking several aspects. First of all the process is reserved for new constructions or constructions subjected to a heavy rehabilitation. The structure in which it is installed must be dimensioned to the withstand earthquake and the reinforcement must be known and able to bear the forces generated by the PIRs. The design of PIRs must therefore be carried out at the same time as verification of the reinforcement of the existing structure. Also all the requirements given in the European Technical Assessment must be kept.
3.1.1 Design value

The goal of this seismic evaluation is to assess whether the PIR can develop similar performance as cast in rebar in concrete in the range of concrete strength C20/25 to C50/60. In low strength concrete EN 1992-2 \(^7\) specify that the design ultimate bond stress of a cast in rebar is equal to \(f_{bd} = 2,3 \text{ N/mm}^2\) for concrete class C20/25. This value is calculated on the basis of Equation 8.2 of EN 1992-2.

\[
f_{bd} = 2,25 \cdot \eta_1 \cdot \eta_2 \cdot f_{cdt}
\]

Where:

\(\eta_1 = 1,0\) for good bonded conditions

\(\eta_2 = 1,0\) for post installed rebar diameter \(\phi \leq 32 \text{ mm}\)

\(\eta_2 = (132 - \Phi)/100\) for post installed rebar diameter \(\phi > 32 \text{ mm}\)

\(f_{cdt} = \alpha_{ct} \cdot f_{ctk,0.05}/\gamma_c\)

\(\alpha_{ct}\) coefficient taking account of long term effects on the tensile strength and of unfavorable effects, resulting from the way the load is applied and the partial safety factor for concrete taken as \(\gamma_c = 1.5\)

In addition the European Assessment Document \(^8\) EAD n°14-330087-00-0601 “Systems for post-installed rebar connections with mortar” specify that in order to be able to publish a design ultimate bond stress \(f_{bd} = 2,3 \text{ N/mm}^2\) the PIR shall exhibit an ultimate mean bond resistance \(f_{b,\text{m,rd}} \geq 10,0 \text{ N/mm}^2\). This value is derived from the design ultimate bond stress \(f_{bd}\) by multiplying with several coefficients that took into account the following aspects:

- Coefficient to pass from the mean to the characteristic value,
- Partial safety factor of the PIR needed to derived the design bond strength,
- Influence of cracks in the concrete,
- Influence of the confinement with tests for the determination of ultimate mean bond resistance,

3.1.2 Methodology for building of importance class up to class III

The goal of this seismic evaluation is to assess whether the PIR can develop similar performance as cast in rebar in concrete in the range of concrete strength C20/25 to C50/60. The pre requisite for a seismic qualification of a PIR is that the bonding element must be qualified according to Technical report TR023 \(^9\) or according to EAD 14-330087-00-0601 \(^8\) that cover the assessment of PIR under static loading in concrete.

Also, as the TR023 and the EAD 14-330087-00-0601 don’t cover the behavior of the mortar in cracked concrete it is necessary to ensure that it is suitable for use in cracks because during an earthquake the structure will be subjected to compression and tension forces that will induced cracks into the concrete where PIR will be installed. Consequently the second pre requisite for a seismic qualification of a PIR is that the polymer resin must be qualified according to ETAG001 Part 5 \(^10\) according to option 1 that cover the assessment of bonded fastener for use in cracked concrete.
In order to evaluate the influence of the compression and tension forces that will be applied to the PIRs during a seismic event, dynamic tests are performed. These tests are identical to the seismic tension tests described in ETAG001 Annex E11 tables 1.1 or 2.1 or Technical Report TR04912 tables 2.1 and 2.4 where pulsating tension load is applied to the rebar embedded in a cracked concrete.

Depending on the importance class of the building where the PIR are installed and of the seismic area where the building is built, the type of seismic test is different and the Table 1 summarizes which test has to be performed.

<table>
<thead>
<tr>
<th>Seismicity</th>
<th>Importance class acc. to EN 1998-1:2004, 4.2.5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class</td>
<td></td>
</tr>
<tr>
<td>Very low¹)</td>
<td>(a_g \cdot S \leq 0.05 \text{ g})</td>
</tr>
<tr>
<td>Low²)</td>
<td>(0.05 \leq a_g \cdot S \leq 0.1 \text{ g})</td>
</tr>
<tr>
<td>&gt; Low²)</td>
<td>(a_g \cdot S &gt; 0.1 \text{ g})</td>
</tr>
</tbody>
</table>

Table 1: Type of seismic tests as function of seismic peak ground acceleration and importance class of building

1) Definition according to EN 1998-1:2004, 3.2.1
2) \(a_g = \gamma_1 \cdot a_{gr}\) Design ground acceleration on type A ground (Ground types as defined in EN 1998-1:2004, Table 3.1); 
   \(\gamma_1\) = importance factor (see EN 1998-1:2004, 4.2.5); 
   \(a_{gr}\) = reference peak ground acceleration on type A ground (see EN 1998-1:2004, 3.2.1); 
   \(S\) = soil factor (see e.g. EN 1998-1:2004, 3.2.2).

On the basis of these tests the seismic design ultimate bond stress \(f_{bd,seis}\) has to be determined. As a first step, the maximum seismic bond strength \(\tau_{eq}\) applied on the rebar is compared to the design ultimate bond stress \(f_{bd}\) of a cast in rebar as calculated in 3.1.1 and summarized in Table 2. In case where the maximum seismic bond strength \(\tau_{eq}\) applied to the rebar is below the design ultimate bond stress \(f_{bd}\) of a cast in rebar for the considered concrete class, then the value of \(f_{bd,seis}\) as to be reduced down to the next concrete class where this criteria is verified.

An example of this comparison is shown on Figure 6 for the concrete compressive class C40/50. For rebar size 32 mm it can be noticed that the seismic bond strength \(\tau_{eq}\) applied is smaller than the required \(f_{bd}\) value specified in Table 2 and that the nearest lower value \(f_{bd}\) is 3.0 N/mm². This verification must be done for each size and concrete class ranging from C20/25 to C50/60.

In a second step, the ultimate residual bond strength determined after the seismic tests \(\tau_{res}\) is compared to the required bond resistance \(f_{bm,rqd}\). When this residual bond strength is below the required bond resistance \(f_{bm,rqd}\) then a reduced value of seismic design ultimate bond stress \(f_{bd,seis}\) is determined taken as the next concrete class where this criteria is verified. As an example this comparison is shown on Figure 7 for the concrete compressive class C40/50.

As an example this comparison is shown on Figure 7 for the concrete compressive class C40/50. For rebar size 32 mm it can be noticed that the residual bond strength \(\tau_{res}\) is smaller than the required \(f_{bm,rqd}\) value specified in Table 2 and that the nearest lower value \(f_{bm,rqd}\) is 10.0 N.mm² which correspond to a minimum value requested for using the design ultimate bond stress \(f_{bd}\) according to
Table 2. This verification must be done for each size and concrete class ranging from C20/25 to C50/60.

Finally the seismic design ultimate bond stress $f_{bd,seis}$ is taken as the minimum value of the seismic design ultimate bond stress $f_{bd,seis}$ determined by first comparing the maximum seismic bond strength $\tau_{eq}$ applied on the rebar and the design ultimate bond stress $f_{bd}$ of a cast in rebar and second by comparing the ultimate residual bond strength determined after the seismic tests $\tau_{res}$ to the required bond resistance $f_{bm,rqd}$. The Figure 8 shows for the above example for size 32 mm the values of $f_{bd}$ determined at the first step is $f_{bd,seis} = 3,0 \text{ N/mm}^2$. The value of $f_{bd}$ determined at the second step is equal to $f_{bd,seis} = 2,0 \text{ N/mm}^2$. Consequently the value of the seismic design bond strength for the size 32 mm and for concrete class C40/50 is the minimum of these two values, $f_{bd,seis} = 2,0 \text{ N/mm}^2$. 

![Figure 6: Comparison of $\tau_{eq}$ with $f_{bd}$](image)

![Figure 7: Comparison of $\tau_{res}$ with $f_{bm,rqd}$](image)

![Figure 8: Minimum value of $f_{bd}$ valid for seismic design](image)

<table>
<thead>
<tr>
<th>Concrete strength class</th>
<th>Required bond resistance for post installed rebar $f_{bm,rqd}$ [N/mm²]</th>
<th>Design value of the ultimate bond stress according to EN 1992-1-1) $f_{bd}$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C20/25</td>
<td>10,0</td>
<td>2,3</td>
</tr>
<tr>
<td>C25/30</td>
<td>11,6</td>
<td>2,7</td>
</tr>
<tr>
<td>C30/37</td>
<td>13,1</td>
<td>3,0</td>
</tr>
<tr>
<td>C35/45</td>
<td>14,5</td>
<td>3,4</td>
</tr>
<tr>
<td>C40/50</td>
<td>15,9</td>
<td>3,7</td>
</tr>
<tr>
<td>C45/55</td>
<td>17,2</td>
<td>4,0</td>
</tr>
<tr>
<td>C50/60</td>
<td>18,4</td>
<td>4,3</td>
</tr>
</tbody>
</table>

Table 2: Required bond resistance
3.1.3 Methodology for building of importance class IV

A differentiation has been made between buildings of importance class up to class III and buildings of class IV because higher importance class buildings require a higher level of safety that is ensured by having more severe testing protocols involving larger cracks and more demanding loading history. Thus in addition to the requirements given in section 3.1.2 the additional tests according to method C2 have to be performed. These tests evaluate the influence of larger cracks as well as cycling cracks opening on the bond strength of the PIRs. This methodology is similar to the one described in ETAG001 Annex E and the reduction factor determined through this analysis will be applied to the design value of the ultimate bond stress $f_{bd}$ determined with method C1. These reduction factors take account of:

- Reduction factors because mean ultimate failure load doesn’t meet the required value: $\alpha_{C2.1a}$, $\alpha_{C2.3c}$ and $\alpha_{C2.5c}$
- Reduction factors because applied load during cycling need to be reduced in order to avoid failure during cycling: $\alpha_{C2.3a}$, $\alpha_{C2.3b}$, $\alpha_{C2.5a}$ and $\alpha_{C2.5b}$
- Reduction factors because the coefficient of the failure load is larger than the required value: $\beta_{cv,L2.1a}$, $\beta_{cv,L2.3}$, and $\beta_{cv,L2.5}$.

The calculation of these reduction factors is given in ETAG001 Annex E as well as in TR049.

The final design bond strength deduced form these tests, is calculated as follow:

$$f_{bd,seis,C2} = \alpha_{N,seis,C2} \cdot \beta_{cv,N,C2} \cdot f_{bd,seis,C1} \text{ with}$$

$$\alpha_{N,seis,C2} = \alpha_{C2.1a} \cdot \min(\alpha_{C2.3a};\alpha_{C2.5a})$$

$$\beta_{cv,N,C2} = \min(\beta_{cv,L2.1a};\beta_{cv,L2.3};\beta_{cv,L2.5})$$

3.2 Tests description

3.2.1 Static tests

As a first step the PIR have to be evaluated under static and quasi static loading. The evaluation is described in EAD 14-33-0087-06.01 “Systems for post-installed rebar connections with mortar”\textsuperscript{8}. This EAD allows to evaluate the post-installed rebar connections under static loading and to assess that their performance is at least equal to the one of the cast in rebar.

The bond strength in low and high strength concrete is verified to be higher than the ultimate mean bond resistance $f_{bm,\text{r,qd}} \geq 10,0 \text{ N/mm}^2$. The influence of the crack is checked by doing tension tests in crack width $w=0.30 \text{ mm}$ and when the influence of the crack for PIR is shown to be higher than the one for cast in rebar then an amplification factor $\alpha_{lb}$ is determined and apply to the designed embedment depth.

In addition the sensitivity to reduced cleaning in dry and wet concrete is tested for different sizes ranging from small to large diameter with a drilling depth equal to $10\phi +300 \text{ mm}$. In case where the sensitivity to reduced cleaning is shown, a reduction factor is applied to the ultimate mean bond resistance $f_{bm,\text{test}}$. 

1182
Correct injection is verified at maximum embedment depth in order to evaluate the feasibility of drilling and cleaning the hole as well as the possibility to fill the bore hole with the mortar from the bottom of the hole up to the concrete surface at both minimum and maximum installation temperature. Presence of voids during the injection process and sagging of the rebar into the hole are verified as well.

The installation with vertical upwards and horizontal directions is checked in order to verify that creep of the rebar during curing time of the mortar doesn’t happen and that leak of mortar out of the hole is prevented.

Furthermore the creep behaviour at normal ambient temperature \( T=20^\circ\text{C} \) and maximum long term temperature \( T=50^\circ\text{C} \) is verified as well as the behaviour of the PIR under freeze / thaw condition. Any reduction of the ultimate pull-out capacity in the residual tension load test leads to the calculation of the reduction factor that will be applied to the ultimate mean bond resistance \( f_{bm,\text{test}} \). The durability of the mortar under alkaline and sulphurous environmental exposure is verified in order that it doesn’t affect the bond strength of the PIR and that the protection of the PIR against corrosion is at the same level as cast in rebar which is protected by the passivation layer due to the cement paste.

### 3.2.2 Dynamic tests C1

In order to assess the performance of the post-installed rebar connections under seismic loading, dynamic tension tests are performed on the bonded element. The minimum number of sizes to be tested in order to estimate the behavior of the PIR under seismic loading is four different sizes. They must be representative of the actual range of rebar covered in the European Technical Assessment (minimum and maximum diameters and 2 intermediate diameters). The minimum number of tests per sizes is at least five replicates. These tests are identical to the tests described in ETAG001 Annex E Table 2.1 line 1 or in TR049 Table 2.1 line 1. They are confined tests performed on PIR. An embedment depth of \( h_{ef} = 7\phi \) is recommended under the condition that no steel failure occurs during testing. In such case the embedment depth should be reduced. The loading history is given in Table 3 and Figure 10.
Table 3: Required loading history for test method C1

<table>
<thead>
<tr>
<th>Load level</th>
<th>( N_{eq} )</th>
<th>( N_i )</th>
<th>( N_m )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of cycles</td>
<td>10</td>
<td>30</td>
<td>100</td>
</tr>
</tbody>
</table>

Figure 10: Load history for test method C1

With:

\( N_{eq} \) chosen by the manufacturer of the polymer resin. \( N_{eq} \) is calculated on the basis of the rebar diameter, embedment length and applied cyclic bond strength \( \tau_{eq} \). According to methodology described in 3.1.2 there is no need to apply a value of \( \tau_{eq} \) larger than highest value of \( f_{bd} \) published in the relevant European Technical Assessment as the value of \( f_{bd,seis} \) is limited by \( f_{bd} \).

\[ N_i = 0.75 \cdot N_{eq} \quad \text{and} \quad N_m = 0.5 \cdot N_{eq} \]

The PIR has to complete the full cyclic loading history successfully. Otherwise a reduced cyclic load \( N_{eq} \) has to be applied. Following the cycling history, a residual load is performed and the mean ultimate load of the test series is compared to the required bond resistance for post installed rebar \( f_{bm,rodl} \).

### 3.2.3 Dynamic tests C2

The minimum number of sizes to be tested is identical to method C1 and they must be representative of the actual range of rebar covered in the European Technical Assessment (minimum and maximum diameters and 2 intermediate diameters). The minimum number of tests per sizes is at least five replicates. The test method C2 is similar to the testing procedure described in ETAG001 Annex E Table 2.4 lines C2.1a, C2.3 and C2.5 or in TR049 Table 2.4 lines C2.1a, C2.3 and C2.5.

Tests according to line C2.1a are confined tension tests in low strength concrete performed in crack width \( w= 0.8 \) mm. they are intended to serve as reference tests in order to check the influence of pulsating tension loads ( tests C2.3) and functioning with tension load under varying crack width (tests C2.5).

Tests according to line C2.3 are performed using a confined test setup, the loading history is given in Table 4 and Figure 11

The cyclic load is increased step by step while the crack opening is varying from 0,5 mm from beginning up to 50 cycles and then is widen to 0,8 mm until the end of the cycling loading. The crack opening in two steps is to represent the serviceability and the suitability state\textsuperscript{13}. 

\textsuperscript{13}
Table 4: Required load amplitudes for test series C2.3

<table>
<thead>
<tr>
<th>N/Nmax</th>
<th>Number of cycles</th>
<th>Crack width Δw [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>25</td>
<td>0.5</td>
</tr>
<tr>
<td>0.3</td>
<td>15</td>
<td>0.5</td>
</tr>
<tr>
<td>0.4</td>
<td>5</td>
<td>0.5</td>
</tr>
<tr>
<td>0.5</td>
<td>5</td>
<td>0.5</td>
</tr>
<tr>
<td>0.6</td>
<td>5</td>
<td>0.8</td>
</tr>
<tr>
<td>0.7</td>
<td>5</td>
<td>0.8</td>
</tr>
<tr>
<td>0.8</td>
<td>5</td>
<td>0.8</td>
</tr>
<tr>
<td>0.9</td>
<td>5</td>
<td>0.8</td>
</tr>
<tr>
<td>1.0</td>
<td>5</td>
<td>0.8</td>
</tr>
<tr>
<td>SUM</td>
<td>75</td>
<td>-</td>
</tr>
</tbody>
</table>

Tests according to line C2.5 are performed using a confined test setup. They are done in crack concrete where the crack width is cycling within an opening range from 0.1 mm to 0.8 mm meanwhile constant load $N_{w1}$ and $N_{w2}$ are applied successively to the PIR. Following the cycling history, a residual load is performed and the mean ultimate load of the test series C2.5 is compared to the mean ultimate load of the test series C2.1a.

Table 5: Required load amplitudes for test series C2.5

<table>
<thead>
<tr>
<th>N/Nmax</th>
<th>Number of cycles</th>
<th>Crack width Δw [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>20</td>
<td>$N_{w1}$</td>
</tr>
<tr>
<td>0.2</td>
<td>10</td>
<td>$N_{w1}$</td>
</tr>
<tr>
<td>0.3</td>
<td>5</td>
<td>$N_{w1}$</td>
</tr>
<tr>
<td>0.4</td>
<td>5</td>
<td>$N_{w1}$</td>
</tr>
<tr>
<td>0.5</td>
<td>5</td>
<td>$N_{w1}$</td>
</tr>
<tr>
<td>0.6</td>
<td>5</td>
<td>$N_{w2}$</td>
</tr>
<tr>
<td>0.7</td>
<td>5</td>
<td>$N_{w2}$</td>
</tr>
<tr>
<td>0.8</td>
<td>4</td>
<td>$N_{w2}$</td>
</tr>
<tr>
<td>SUM</td>
<td>59</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 11: Schematic test procedure C2.3

Figure 12: Schematic test procedure C2.5
4 Conclusion

This paper presents the methodology used by the Centre Scientifique et Technique du Batiment for the assessment of PIR under seismic loading which is commonly used for national evaluations. The method includes the prerequisite qualification of the PIR under static loading according to EAD 330087-00-0601 as well as the qualification of the polymer resin for a suitable use in cracked concrete according to ETAG001 Part 5. This ensures that the PIR can be designed according to EN 1992-1-1 (Eurocode 2). The seismic evaluation is then done by performing seismic tension tests in cracked concrete. The comparison of the cycling load applied during cycling to the design bond strength $f_{bd}$ as well as the residual load to the required bond resistance for post installed rebar $f_{bmm,reqd}$, allow to verify that the PIR has the same level of performance as the cast in rebars. In addition when the PIR is intended to be used in buildings of importance class IV an additional testing protocol is performed which is more demanding for the rebar. Then a potential additional reduction factor is applied to the designed bond strength. This evaluation method is valid for France and it could be possible to extend this scientific background to other European countries by working on the new European Assessment Document that could make available for the Technical Approval Bodies to issue European Technical Assessments.

References:


8. EOTA, EAD 330087-00-0601, Systems for post-installed rebar connections with mortar, July 2015.


ADVANCES IN CONSTRUCTION /
DEVELOPMENT METHODS
A SURVEY ON THE INITIATION OF COMPUTATIONAL METHODS IN ANCHOR DEVELOPMENT PROCESSES

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ABSTRACT

Past research publications on the utilization of the finite element method (FEM) for anchorages in concrete rely very often on advanced concrete models. They have been programmed and developed within extensive research projects. This contribution shows how the finite element method can be established as a worthy tool within an anchor development process. In this paper a survey will be outlined how reliable calculations for concrete anchors can be performed with a commercially available explicit finite element program.

Within this survey the most important model parameters like load application, the loading velocity, boundary conditions, the time step control or the finite element formulation will be discussed. Further, the incorporated phenomenological plasticity concrete model with damage formulation will be presented. Emphasis will be put additionally on the calibration of the finite element model based on empirical results from literature and existing design models. It will be shown that in a first approach the results are strongly dependent on the applied mesh size. However, by utilizing Bazant’s crack band theory a constant fracture energy throughout the model is applied and objectiveness for the finite element results can be guaranteed.

Finally, a meaningful example will demonstrate that the finite element model can successfully accompany the development process of cast-in-place anchors.

1 Introduction

Nowadays, computer aided engineering (CAE) is state of the art in advanced product development. The list of advantages for the product is long: most importantly, design engineers gain a deeper insight in the stress distribution of the parts, the fatigue life can be predicted, the structural topology can be optimized, the crash performance can be evaluated and the development cycles can be accelerated.

The demands on the computational methods are high as finite element analysis (FEA) for anchorages has to deal with highly nonlinear, three dimensional problems. These nonlinearities arise from the geometry itself, the contact formulation or the incorporated materials like steel, plastics or concrete. An important feature of the concrete material model for anchor simulations is the capacity to show concrete cone failure through cracking. There are various approaches to capture the failure behavior
of concrete material. Hofstetter and Meschke\textsuperscript{1} give an comprehensive overview. Jirásek\textsuperscript{2} showed that it is possible to work with damage and smeared crack models. In this sense, Ottosen\textsuperscript{3}, Han and Chen\textsuperscript{4} or Schütt\textsuperscript{5} modelled the crack by means of damage variables or crack variables. The information is stored in each finite element in each integration point. Then, one integration point represents the smallest amount of cracked material. Huespe and Oliver\textsuperscript{6} described an algorithm for failure analysis with strong discontinuities named Continuum-Strong Discontinuity Approach (CSDA) which is an enhancement of the damage and smeared crack models. To represent the crack in a discrete manner, the finite element mesh has to separate at the location of the crack and adaptive remeshing has to be performed to calculate arbitrary crack paths. Further, the concrete material can be modeled on a meso-level (Carol, et al.\textsuperscript{7}) for which cracks are simulated with interlace element of zero thickness. This approach separates the aggregate from the cement matrix by a distinct finite element mesh. Last but not least, the extended finite element method (XFEM) can be used for the modeling of cracks. This technique enriches the ansatz functions to capture the discontinuities of a crack within a finite element. According to Moes\textsuperscript{8} this approach was first introduced by Melenk and Babuska\textsuperscript{9} for 2D applications. Loehnert, Mueller-Hoeppe and Wriggers\textsuperscript{10} expanded the method to 3D applications.

Formulating the computational investigation of anchorage requires a deep insight into load carrying theory and in existing test results. Eligehausen, Mallée and Silva\textsuperscript{11} collected the data published in this research field which originated to a great extent from the Department of Fastening Technology of the University of Stuttgart. The investigation of cast-in-place anchors plays a key role. In particular, headed studs are probably the most investigated anchor type. Furche\textsuperscript{12} examined axially loaded headed studs and Zhao\textsuperscript{13} extended the studies to anchor groups and anchors loaded in shear direction. Hofmann\textsuperscript{14} focused on shear loaded anchors and concrete edge failure and compared the experimental test data to numerical simulations. Here, the incorporated numerical tool is called “Macroscopic Space Analysis” (MASA). It was developed by Prof. Josko Ozbolt et al. (Appl\textsuperscript{15}). MASA uses the microplane model for the concrete material in order to receive objective and mesh independent results (Bazant and Ozbolt\textsuperscript{16}, Ozbolt and Eligehausen\textsuperscript{17}) and to represent size effects (Bazant and Planas\textsuperscript{18} and Ozbolt\textsuperscript{19}). Further, renowned manufacturers use high performing in-house software for the detailed analysis of their products (Li, et al.\textsuperscript{20}, Li\textsuperscript{21}).

This contribution focuses on the validation process for a simulation setup for cast-in-place headed studs. In particular, a benchmark example of an axially loaded headed stud is investigated and compared to empirical solutions taken from literature. The explicit finite element analysis program RADIOSS from Altair Hyperworks enables simulations with large displacements. Moreover, a dynamic solver allows earthquake analysis for future projects and provides a fast integration algorithm for the consistent material tangent. But, the input parameters for the simulation have to be chosen carefully. Seven aspects were found to be critical for a reliable and reproducible anchor simulation. Several iterations over those points are needed to find the optimal setup (Figure 1):
i. Time step control,
ii. Load application and loading velocity,
iii. Element formulation,
iv. Boundary conditions,
v. Material model and its parameters,
vi. Mesh size,
vii. Crack Band Method.

In what follows, each of the above aspects is discussed in consecutive order, and its influence and sensitivity on the result studied for the example depicted in Figure 1.

2 Core parameters for quasi-static anchor simulations

2.1 Time step control

Explicit finite element simulations discretize the problem in space and in time. While discretization in space refers to the finite element mesh, time stepping discretizes the run time into a whole lot of equilibrium iterations. Generally, the time step should not exceed the duration that sound needs to cross an element of a specific material. The minimum value of the element or nodal time step

\[ \Delta t = \min \left( \frac{l_c}{c}, \sqrt{\frac{2M}{K}} \right) \]  

becomes critical. In equation (1), \( l_c \) represents the characteristic length, \( c \) represents the speed of sound in a solid material with \( c = \sqrt{\frac{E}{\rho}} \), \( M \) is the concentrated nodal mass and \( K \) is the equivalent nodal stiffness. \( E \) is the Young’s modulus and \( \rho \) is the density of the material. There are several possibilities to increase the time step for calculation time optimization. However, it is advised to leave it on its natural level to avoid unnecessary extrapolation errors.

2.2 Load application and loading velocity

A rigid body placed at the top of the anchor shaft allows for the application of uniform displacement and velocity boundary conditions. An imposed velocity on the master node of the rigid body defines the constraint. The corresponding force is calculated vice versa. To avoid extreme accelerations in the system a sinusoidal step function is used from the start, see Figure 2. It is followed by a constant imposed velocity. The value of the constant velocity has to be chosen carefully. Any effects of inertia have to approach zero. A convergence analysis has to be conducted with varying velocities. A quasi-static solution is found iteratively. The overall load-deflection-behavior must not be affected by the loading velocity. For the example given in Figure 1, the anchor is accelerated in \( t = 0.001 \, s \) to the constant velocity \( v = 10 \, mm/s \).
2.3 Element formulation

The finite element model is put up with four node isoparametric tetrahedral elements (Figure 3). For the integration of the linear ansatz functions one Gauß point is used. Thus, the finite element is fully integrated, see RADIOSS Theory Manual\textsuperscript{22}. The use of tetrahedral elements yields potentially four times more elements than a hexahedral finite element discretization when the number of nodes is kept constant. But, only half the number of integration points is needed in comparison to fully integrated hexahedral elements. This choice of a finite element circumvents the draw backs of underintegrated elements and possibly the presence of hourglass energy in the problem. Further, tetrahedral elements place the analyzing engineer in the situation of easy meshing. For future complicated anchor geometries this is seen as a great advantage.

2.4 Boundary conditions

The definition of suitable boundary conditions plays a critical role for anchor simulations (Hofmann\textsuperscript{14}).

With an effective embedment depth $h_{ef}$ of the anchor, supports are placed radially around the longitudinal axis in the distance $d = 3 \cdot h_{ef}$ on top of the concrete block. These nodes are fixed in all three degrees of freedom. Additional concrete volume with a depth $h_c = 3.5 \cdot h_{ef}$ below the anchor...
prevents the specimen being split when the anchor is pulled out. It may be assumed that the results are not influenced by the boundary conditions (Ozbolt and Eligehausen17).

Figure 4: Von-Mises and Druck-Prager surface in principal stress space

Further supports are attached to the concrete along the lower third of the anchor shaft in order to account for the confinement effects arising from the steel anchor in the circumference of the anchor head. The radial degree of freedom is suppressed.

The connection between the head of the anchor and the concrete is realized via node to node meshing. Using symmetry boundary conditions cuts the model to a fourth of its original size, thus accelerating the calculation time substantially.

2.5 Material model and its parameters

The given example consists of steel and concrete. The steel material is represented by a simple elastic-plastic hardening law of von Mises type (Figure 4). Five material parameters are sufficient to describe the full three-dimensional behavior. These are the density $\rho$, the Young’s modulus $E$, the Poisson ratio $\nu$, the hardening modulus $H$ and the yield stress $\sigma_y$ (Simo and Hughes23). The concrete behavior is substantially more complex. The derivation of the material model originates from the definition of the failure surface. Here, the Ottosen failure surface (Ottosen3) is implemented. The failure surface is defined in 3D principal stress space. As a surface of Drucker-Prager type (Figure 4), it has a conical shape with a closed cap in the tensile regime and an open end in high hydrostatic pressures. In radial direction it has a slightly rounded triangular shape, see i.e. (Speck24). The yield surface is derived from the failure surface by downscaling it with a nonlinear function $k(\sigma_m)$ dependent of the hydrostatic stress $\sigma_m$. For tensile hydrostatic stressstates the failure surface and the yield surface are congruent while the yield surface has a closed cap for high hydrostatic pressures in contradiction to the failure surface which is open.
A trilinear stress-strain-behavior for the concrete can be found for uniaxial compression. The first kink equals a stress-state reaching the yield surface, and the second kink leads into an ideally plastic material answer and is connected to the failure surface (Figure 5 a)). Also the uniaxial tensile behavior is approximated by a trilinear stress-strain-curve (Figure 5 b)). The stress increases linearly with increasing strain until it reaches its maximum where material disintegration evolves and results in degradation of stiffness. The degree of material damage is described by the damage factor $\alpha$ which is 0 at the onset of the damage process and gradually increases to approach the value of 1 when the material fully damaged. Once the specified failure strain $\epsilon_{\text{max}}$ is reached, the finite element fails and is deleted. The material model in use offers the possibility for a very detailed and exact analysis with 21 material parameters. Anyway, it was found that very good results can be found with default parameters after inserting five core parameters (Martin, Centro and Schwoertzig25). These are namely the material density $\rho$, the Young’s modulus $E$, Poisson ratio $\nu$, uniaxial compression strength $f_c$ and the tangent tensile modulus $H_t$.

### 2.6 Mesh size

The FEM is an approximation method. The accuracy of the solution improves with a finer mesh. However, more elements result into an increased calculation time. An optimum has to be found to balance accuracy of solution and required computational resources.

When damage formulations are used, another phenomenon can be observed. The analysis results in lower ultimate strengths for finer meshes. This is caused by the localization of damage, and ultimately the crack, in element bands for which less energy has to be dissipated when the mesh is finer and the element size smaller (Grassl, et al26, Dimitrijevic and Hackl27, Jirasek2). This effect can be counteracted by adjusting the material parameters to a mesh of constant element size. Because concrete fails primarily due to high tensile strains, the relevant material parameters are the Young’s modulus $E$, the tensile strength $f_t$ and the tangent tensile modulus $H_t$. Since the Young’s modulus is linked to the definition of the compression and tension domain, the tensile strength is generally set to 10 % of the compression strength, the only free parameter remaining is $H_t$. Thus, the softening modulus $H_t$ can be used to adjust the fracture energy $G_f$ and to find the pull-out load for the anchor.

### 2.7 Crack Band Method

In Bazant and Oh28 the fracture energy

$$ G_f = (2.72 + 0.0214 \cdot f_t) \cdot f_t^2 \cdot \frac{d_a}{E} $$

(3)

is related to the Young’s modulus $E$, tensile strength $f_t$ and the biggest aggregate size $d_a$. With this fracture energy and an effective element size $h$, the softening modulus

$$ H_t = \left( \frac{1}{E} - \frac{2G_f}{f_t^2h} \right)^{-1} $$

(4)

can be calculated. Equations (3) and (4) are unit sensitive and are true for psi unit system. By setting the corresponding material parameters for each element size, variable element sizes can be used without suffering from mesh dependent results or localization effects. This allows to reduce the number of elements which can be chosen small in the areas of interest (conical crack, micro cracks at
the top of the concrete in the circumference of the shaft) and large in all other areas. A performant script for the process automation is desirable.

3 Validation of the survey

This section is to demonstrate the ability to yield meaningful finite element results by using the rules of action presented in the above survey. The example of a headed stud will be analyzed and the FEM results will be compared to results based on the widely recognized empirical design equation

\[ N_{Rk}^c = 15.5 \cdot f_{cc,200}^{0.5} \cdot h_{ef}^{1.5} \]  

(5)

for axially loaded single anchors (Pregartner²⁰, Eligehausen, Mallée and Silva¹¹). The uniaxial concrete compression strength of a concrete cube with side length \( a = 200 \text{ mm} \) is represented by \( f_{cc,200} \) and the effective embedment depth by \( h_{ef} \). For the presented example of a headed stud embedded in an uncracked concrete block, the effective length measures \( h_{ef} = 80 \text{ mm} \). Thus, the predicted pull-out load for concrete with \( f_{cc,200} = 29 \text{ MPa} \) equals \( N_{Rk}^c = 15.5 \cdot 29^{0.5} \cdot 80^{1.5} \cdot 0.001 = 59.7 \text{ kN} \). The geometry of the concrete block is depicted in Figure 6 a) and the parameters for the material mode are collected in Table 1.

3.1 Iteration of the softening modulus

As stated in section 2.6 the softening modulus \( H_t \) becomes a free parameter to adjust the fracture energy. For an iteration of the correct pull-out load according to equation (5) the softening modulus is varied. The failure strain

\[ \varepsilon_{\text{max}}(H_t) = \left| \frac{f_t}{E} \right| + \left| \frac{f_f}{H_t} \right| \]  

(7)

is calculated as a function of \( H_t \). All other parameters are left in default state (Table 1). The area of a potential crack is modeled with elements of a constant mesh size \( a = 2 \text{ mm} \) (Figure 7 a)). An iteration like this yields a tangent tensile modulus \( H_t \approx -250 \text{ MPa} \), Figure 8.

Figure 6: Example model of a headed stud in a concrete block a) Geometries, b) Finite element mesh
Table 1: Concrete material parameters

<table>
<thead>
<tr>
<th>Material Parameter</th>
<th>Value</th>
<th>Unity</th>
<th>Material Parameter</th>
<th>Value</th>
<th>Unity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density ( \rho )</td>
<td>2.50E – 09</td>
<td>( \frac{t}{mm^3} )</td>
<td>Young’s modulus ( E )</td>
<td>210000</td>
<td>MPa</td>
</tr>
<tr>
<td>Poisson ratio ( \nu )</td>
<td>0.2</td>
<td>[-]</td>
<td>Confined strength ratio ( \frac{f_s}{f_c} )</td>
<td>4.0</td>
<td>[-]</td>
</tr>
<tr>
<td>Compression strength ( f_c )</td>
<td>29.9</td>
<td>MPa</td>
<td>Confining stress ratio ( \frac{s_0}{f_c} )</td>
<td>1.25</td>
<td>[-]</td>
</tr>
<tr>
<td>Tensile strength ratio ( \frac{f_t}{f_c} )</td>
<td>0.1</td>
<td>[-]</td>
<td>Maximum damage ( D_{SUP} )</td>
<td>0.9999</td>
<td>[-]</td>
</tr>
<tr>
<td>Biaxial strength ratio ( \frac{f_b}{f_c} )</td>
<td>1.2</td>
<td>[-]</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 7: Mesh renderings: a) constant mesh size \( a = 2 \ mm \) in the crack region, one set of material parameters, b) crack development for \( a = 2 \ mm \), c) arbitrary mesh size using crack band method, color code indicates the different material sets, d) crack development for crack band method

Figure 8: Influence of the tensile tangent modulus on the load-deflection-behavior
3.2 Crack Band Method

The crack band method will be applied to the example. The constant element size \( a = 2 \, \text{mm} \) remains unchanged. It is assumed that the height

\[
h_{\Delta} = \frac{a}{3} \cdot \sqrt{6} = 1.633 \, \text{mm}
\]  

(8)

of the tetrahedral finite element equals the effective crack band width. Provided a greatest grain size \( d_a = 8 \, \text{mm} \), this yields a softening modulus \( H_t = -267.92 \, \text{MPa} \) in very good accordance to section 3.1. Also, the load-deflection curves in Figure 8 proof the validity of this approach. These considerations result in a fracture energy \( G_f = 0.0289 \, \frac{\text{N}}{\text{mm}} \).

Next, a script (cp. section 2.7) is used to determine the actual size of each finite element and assign the corresponding material parameters. At first, the same mesh as before is used (Figure 6 b), Figure 7 a)). 57 equidistant classes of concrete materials are created. The classes range from element sizes \( h = 0.25 \, \text{mm} \) to \( h = 14.5 \, \text{mm} \). Then, a finite element model is discussed which is much coarser (Figure 7 c)). It consists of 90296 nodes in comparison to 617739 nodes. Again, the automated script is used to prepare the model for varying material parameters in order to ensure a constant fracture energy. 86 material classes are created. They are bonded by \( h_{\text{min}} = 0.5 \, \text{mm} \) and \( h_{\text{max}} = 43.5 \, \text{mm} \). Figure 10 shows the resulting load-displacement-curves in very good accordance. However, the cracked region can be captured in more detail with a finer mesh (Figure 7 b), d)).

![Figure 9: Different embedment depths a) \( h_{ef} = 40 \, \text{mm} \), b) \( h_{ef} = 80 \, \text{mm} \) and c) \( h_{ef} = 120 \, \text{mm} \)](image-url)
3.3 Variation of embedment depth

For a final verification the influence of the embedment depth is evaluated by another two simulations with \( h_{ef} = 40 \) mm and \( h_{ef} = 120 \) mm (Figure 9). A constant mesh size and a softening modulus of \( H_t = -250 \) MPa is used. The maximum failure strain is set to \( \varepsilon_{max} = 0.01169 \). The resulting load-displacement-diagram is given in Figure 11 a). A comparison of the ultimate capacity for variable embedment depths with the empirical design equation (5) shows a good accordance (Figure 11 b)).

4 Summary

This contribution provides some guidance for the simulation of concrete anchors. An explicit finite element solver is used to solve the dynamic problem. This allows the user to assess problems with large displacements. It is shown that the presented approach matches empirical results very well. Additionally, advice is given how to get rid of the frequently observed mesh dependence. While this procedure yields good results regarding the pull-out load, the cracked area is better represented when using a finer mesh. It is also shown that the approach can be extrapolated for other embedment depths.
This work focused on the prediction of the ultimate load which according to Eligehausen, Mallée and Silva\textsuperscript{11} can be found by a proper representation of the tensile property of the concrete. In order to capture the deformation behavior accurately, the compression property has to be investigated in more detail.

The views expressed in this paper are the views of the authors only and do not necessarily reflect the views of Stanley Black & Decker, Inc.

References


AN AUTOMATED PRE- AND POST-PROCESSOR FOR THE ANALYSIS OF STEEL COLUMN BASEPLATE CONNECTIONS

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ABSTRACT

Local-level analysis of steel column baseplate connections is critical to the design process, both for the prediction of the overall behavior of the steel structure and for the derivation of design concrete anchor loads. However, existing software used for this task may be limited by certain assumptions used to simplify the solution, such as the assumption of a rigid baseplate, assumed material response data, or by a lack of verification with respect to available connection test data.

This paper presents an automated pre- and post-processor capable of mesh generation, submission, and data postprocessing for the analysis of baseplate connections subjected to lateral loading. The connection is modeled using shell elements for steel components and nonlinear springs for grout and anchor elements. The processors are intended to be used in conjunction with any general-purpose finite element software, such as ABAQUS or LS-DYNA. The preprocessor draws from a unique database of anchor force-deformation test data, allowing accurate prediction of the evolution of anchor forces as well as the overall connection moment-rotation behavior. The mesh generation approach, required inputs, and analysis results are discussed.

An extensive verification effort involving the comparison of the model to approximately 30 baseplate connection tests is first summarized. Subsequently, several parametric analyses are presented and compared to the results of other anchor design-oriented commercial programs to elicit the viability of the design assumptions adopted when determining anchor forces (e.g., rigid baseplate). The accuracy of current anchor design-oriented programs is evaluated and situations in which inaccurate design anchor forces may be produced are highlighted.

1 Introduction

Detailed analysis of baseplate connections during the design process may be necessary for several reasons. For example, connection analysis is necessary to determine the nonlinear connection response for use in the design of the structure as a whole and when calculating anchor forces for use
in anchor selection and design. There are a number of methods currently available to the analyst for this purpose, including:

1. “Hand” analysis methods (e.g. Eurocode EN 1993-1-8:2005\(^1\), American Institute of Steel Construction Design Guide \(^2\))

2. Commercial software, (e.g. PROFIS\(^3\), RISABase). This category includes specialized baseplate connection design/analysis programs and/or anchor design software

3. Custom modeling using general-purpose finite element software

Of these, (1) is often the simplest, however it provides the least amount of information regarding the nonlinear behavior of the connection. This type of method may not be suitable for connections with unusual geometry, anchor layout, or other features. There are several programs which fall into category (2), and they generally provide fast, accurate, and well-organized models of nearly any standard baseplate connection. Such programs generally use relatively simple shell-type models to allow rapid iteration in the design space. However, some commercial software relies on more comprehensive simplifying assumptions, such as the assumption of a rigid baseplate to allow the rapid parametric solution of the problem. As shown later in this paper, the results of such analysis may give inaccurate results for certain combinations of anchor size and baseplate thickness. Additionally, such software is generally not verified in the open literature, and the assumptions that go into the solution are not always made explicit to the user. An example of such an implicit assumption is the nonlinear force-displacement curve of the anchors, which is generally a function of size, grade, and stretch length\(^a\). Incorrect assumptions may have a pronounced effect on the predicted connection behavior. Method (3) involves the development of a full finite element, often starting from a CAD representation of the connection. This method may be used to analyze connections in any conceivable configuration, and allows full control over the model inputs. This type of model is generally very detailed, including either a relatively fine shell element or solid element representation of the baseplate and column, and is favored by researchers and academics. Due to the level of control afforded, such models can be well-verified and demonstrated to be robust by comparing with test data. However, the creation process is generally very time consuming and too expensive for most typical design situations.

2 Program Organization and Data Sources

2.1 Program objectives

As part of a larger investigation into the behavior of steel column baseplate connections, specialized pre- and post-processors were developed to combine the rapid parametric analysis capabilities of commercial software with the modeling accuracy and verifiability of custom models. The objectives of this effort included the following:

1. Use of a well-validated modeling methodology for steel baseplate connections subjected to axial and moment loading

\(^a\) The length over which plastic deformation is expected to occur
2. Automated model generation with minimum input from the user.

3. Fully automated pre-processing, including general model layout, model meshing, and calculation and/or lookup of other parameters such as anchor force-deformation relationships.

4. Automated postprocessing for quantities of typical interest, including connection moment-rotation response, anchor forces and deformations, and baseplate and column stresses

2.2 Program organization

The program consists of two main modules, namely, a preprocessor and a postprocessor. Both modules were written in Python, a common, open-source interpreted language, such that continuous improvements and open sharing of the program is facilitated. A schematic of the typical workflow is shown in Figure 1. First, the preprocessor reads a simple text input file which contains basic information such as the connection geometry and anchor layout, baseplate thickness, column shape, anchor grade, anchor stretch length, and connection loading. Loading may consist of axial, moment, and shear loads, although the modeling methodology has only been verified for axial and moment loads with relatively low shear. The loads may be uni- or multi-directional. Loading may be specified as force- or displacement-based, and may be monotonic or cyclic. Several common displacement-based standardized cyclic seismic load histories are pre-programmed into the script and may be selected by the user. The preprocessor then matches the specified anchor size, grade, and stretch length to a database of anchor test data\(^4\) to determine the anchor force-deformation curve which will be used in the analysis. If the anchor parameters fall outside the bounds of existing test data, interpolation/extrapolation schemes are used to estimate the anchor response.

A typical mesh for a basic 4-anchor connection with a wide-flange column is shown in Figure 2. The model consists of a shell representation of the baseplate and bottom of the column stub. The column stub is attached to a beam-element representation of the upper column with a kinematic coupling at a height selected by the user. The grout and anchors are modeled with nonlinear springs that are compression-only and tension-only, respectively. A full description of the model features shown in Figure 2 is outside the scope of the current paper, but a detailed description of the model is available in the literature\(^5\). To create the model, a meshing algorithm first generates the main baseplate and column stub mesh on a rectangular grid. Various details of this mesh may be adjusted by the user, including the target element size and maximum aspect ratio, as well as the choice of rigidizing elements at the anchor attachment points or at the column-baseplate (BP) intersection. Next, the remainder of the model, including the rest of the column and the anchor and grout springs, is generated and attached. The model typically includes beam elements for the column up to 2/3 of the story height, a location commonly assumed for the inflection point of a laterally-loaded frame.

Following successful meshing, a submission batch file is generated for the analysis program of choice. The output database is monitored for successful completion, and then a postprocessing script interacts with the FEA postprocessor to collect and plot results of typical interest, such as the moment-rotation response of the connection and anchor forces.
Christopher Trautner, Tara Hutchinson, and Roberto Piccinin

Figure 1: Program layout and workflow

- Text Input File
  - Geometry & layout
  - Material properties
  - Loading
  - Selection of analysis program (i.e. LS-Dyna®)

Input file creation
- Meshing algorithm
- Response curve interpolation
- Solution and output requests

Model submission
- Submission executable
- Job monitoring (if required)

Output database

Anchor and grout behavior database
- Anchor force-deformation relationship
- Grout force-deformation relationship

Postprocessing script (with FEA postprocessor)
- Moment-rotation curve
- Anchor forces, deformations
- Baseplate stress, strain

Figure 2: Typical model (model visualized in LS-PrePost for LS-DYNA®)
3 Program Verification

The general analysis method was verified using test data from five different experimental programs, which included a range of connection sizes and parameters, including column section type, baseplate thickness, anchor size and grade, and level of compressive axial load. The experiments included a range of behavior, ranging from “plate-dominated” (thin-plate) connections to connections with very thick plates whose nonlinear behavior was dominated by that of the anchors. The test programs also included a range of failure mechanisms, including baseplate yielding, anchor fracture, weld fracture, and grout compressive failure. Although a full discussion of the verification program is outside the scope of the current paper, this information is available in the literature5.

The modeling approach was found to accurately predict the behavior of a wide range of connections. Figure 3 shows moment-rotation data from the experiments compared with the response calculated using models developed with the current pre-and post-processors. The connections shown range from relatively small, thin plate HSS (Figure 3a) to relatively stout wide-flange section connections with significant axial load (Figure 3b). In both cases, the models show good agreement with the test data.

Figure 3c shows a connection whose ultimate failure mode was concrete breakout. The current model does not include any automatic considerations of concrete-related failure mechanisms; therefore, the data in this plot was truncated at a point at which the anchor force exceeded the anchor group breakout strength, as calculated by ACI 3187. Future efforts may allow the model to capture concrete-dominated failure mechanisms. In contrast, Figure 3d shows a comparison of the calculated response to a connection where failure was caused by anchor fracture. In the case of anchor fracture, the solution process automatically terminates at the point of anchor fracture, as the anchor force drops to zero at the expected anchor ultimate elongation. This singularity causes the implicit solver to stop. As shown in the figure, the agreement of the two curves up to and including fracture is very good.
Figure 3: Hysteretic response predictions for cases with and without significant axial

4 Application and comparison to other software

Several pieces of commercial software used for the analysis of baseplate connections make the assumption of a rigid baseplate on an elastic halfspace to simplify the calculation of anchor forces. The equilibrium and compatibility equations that arise using this assumption are shown in Figure 4 for a unidirectional moment loading. Clearly, this approach is attractive due to its simplicity. Anchor forces may be calculated on the basis of linear equations that do not require an iterative solution. Therefore, the solution of these equations may be carried out almost instantaneously to explore a design space of anchor sizes, anchor layouts, and baseplate parameters.
Although this solution approach has the virtue of simplicity, the applicability of the rigid baseplate assumption is questionable in several common situations. The most obvious situation for which it may not be applicable is for connections with relatively thin baseplates. Most standards and design guides (e.g. Eurocode 3\textsuperscript{1}, AISC DG 1\textsuperscript{2}) have provisions to account for the flexibility and/or strength of the concrete, grout, and baseplate in the calculation of the compression resultant $C$ and the anchor forces. Depending on the rigidity of the baseplate and the strength of the supporting grout, the location of $C$ may be significantly different than predicted by the elastic analysis, changing the moment equilibrium equation and the calculated fastener forces. This potential source of error may be exacerbated by the presence of multiple columns of fasteners (i.e. more than two fasteners into the page for each row shown in Figure 4). Whereas a perfectly rigid baseplate transmits equal force to each of the anchors, a flexible baseplate may transmit significantly more force to the center anchor.

To evaluate this analysis method versus the current finite-element analysis method, two example baseplate configurations were analyzed (Figure 5). Each was 460mm square, supporting a W200x71 (W8x48 US) column. Both utilized 19mm diameter anchors with a nominal yield strength of 380MPa and a nominal ultimate strength of 520MPa, corresponding to a high-strength, low alloy grade common in the United States (ASTM F1554 Grade 55\textsuperscript{11}). The nonlinear force-deformation response of the anchor was measured using a tensile test specimen with a stretch length of 8 in. (Figure 6a). Connection (a) incorporated four anchors on a 330mm grid, while connection (b) utilized six anchors in two rows. Connection (a) is very similar to the connection analyzed in Figure 3b. In both cases, the anchors were assumed to have a 200mm stretch length. The connections were assumed to be set on grout with a 62MPa compressive strength. For simplicity, the two connections were subjected to a monotonic enforced column displacement in the x-direction generating a unidirectional moment about the y-axis about the strong axis of the column such as that shown in Figure 4. Each model was analyzed with five different baseplate thicknesses, ranging from 6mm to 125mm. The thickest of this range is obviously impractical; however it may approximate the response of a stiffened baseplate. The total time required to set up, run, and postprocess the data for

$$C = T1 + T2$$

Figure 4: Equilibrium for rigid baseplate assumption (courtesy: Hilti AG)
all models was on the order of one hour. The moment-rotation curves for the connection shown in Figure 5a are shown in Figure 6b. The curves for the connection shown in Figure 5b were similar, with additional features due to different plastic hinge locations and non-uniform yielding of the anchors (discussed below). The jumps in the 125mm curve are due to the knife-edge effect of the very thick baseplate, which crushed localized areas of the grout suddenly.

![Figure 5: Example baseplate connection configurations](image)

![Figure 6: Selected model input and outputs](image)

Figure 5 shows the anchor tensile force plotted as a function of the applied moment for the connection shown in Figure 5a. The solution from a commercially-available baseplate/anchor analysis software that uses a rigid baseplate (RBP) assumption is plotted at the intersection of the
two dashed red lines, at a design moment of 85 kN-m and an anchor force of about 115 kN. For this particular case, the rigid baseplate assumption appears to work quite well. The 6 and 13mm baseplate connections never reach the design moment, as the baseplate develops full plastic hinges at about 15 and 50 kN-m, respectively. Although such thin baseplates would likely be eliminated from consideration early in the design process, this result illustrates another advantage of the current method, namely the elimination of inadequate designs because of the greater amount of information available from the analysis. Of the connections with practical baseplate thicknesses, only the $t_b=19$mm connection exceeded the anchor force calculated with the rigid baseplate assumption, and then only by about 5%. Using linear interpolation at the design moment, the current model predicted anchor forces equal to those from the rigid baseplate model at $t_b \approx 21$mm.

The analysis with the rigid baseplate is significantly less accurate for the connection shown in Figure 5b. The moment versus anchor force curves for this analysis are shown in Figure 8. In this case, the predicted anchor force is about 77 kN at the design moment. Again, both the 6mm and 13mm baseplates fail to develop the design moment; however, there is a broad band of reasonable baseplate thicknesses that result in anchor forces significantly above those predicted by the RBP analysis at the design moment. As a further illustration of the types of information that are readily available with the parametric analysis, the secant rotational stiffness of both connections is plotted as a function of baseplate thickness in Figure 9. This parameter is of particular use if the baseplate is idealized as a rotational spring in a global structural model, a common practice. Interestingly, at reasonable baseplate thicknesses, the rotational stiffness of the connection is not particularly sensitive to the number of anchors, particularly at larger rotations. This suggests that the estimation of a rotational stiffness for a global model does not necessarily need to be an iterative process.

The anchor forces at the design moment are summarized in Table 1. As may be expected, the exceedances are significantly worse for the middle anchor, up to a maximum of 35%. Neglecting concrete failure modes, which are outside the scope of this study, this result in itself is not particularly concerning. The parametric analysis indicates a positive slope beyond the design moment, indicating that the connection has reserve capacity in excess of the design moment. However, the under-prediction of design anchor loads could result in inadequate design for brittle failure modes such as concrete breakout or side blowout, if the designer were to select anchor embedment or edge distances based on the inaccurate lower loads. This result indicates the need for analysis software to be well-verified and well-vetted for commonly-encountered design situations, to assure that unconservative anchor loads are not used in design.
Figure 7: Individual anchor tensile force versus applied moment as a function of baseplate thickness (connection shown in Figure 5a)

Figure 8: Individual anchor forces from the (a) middle anchor and (b) outer anchor tensile force versus applied moment as a function of baseplate thickness (connection shown in 5b)
Figure 9: Connection rotational stiffness as a function of baseplate thickness

Table 1: Summary of anchor forces at design moment of 85 kN-m

<table>
<thead>
<tr>
<th>Baseplate thickness $t_b$ (mm)</th>
<th>Anchor force from FEA (kN)</th>
<th>Maximum difference to RBP model $(T_{\text{model}} - T_{\text{RBP}})/T_{\text{model}}$</th>
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<tbody>
<tr>
<td>6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>13</td>
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NOTE: the current model predicts anchor forces equal to the rigid baseplate assumption at about 24mm for the outer anchors and about 75mm for the center anchor.

5 Conclusions

A specialized pre- and post-processor for the rapid parametric analysis of steel column baseplate connections is presented. In contrast to existing commercial software, the preprocessor draws upon anchor force-deformation curves generated from a robust tension test database, and the modeling methodology is validated using base connection experiments of the authors and others open literature. To illustrate the usefulness of the presented modeling tool and contrast them with other programs used for the calculation of anchor forces, two example problems were analyzed using the current method and a commercial program that incorporates a rigid baseplate assumption, a common design simplification. The accuracy of the anchor forces was found to be strongly dependent on the thickness of the baseplate. The rigid baseplate analysis software was shown to be reasonably
accurate, however significantly unconservative anchor forces for some reasonable design choices \((t_b \approx 25\text{mm})\) were found. For a similar level of effort from the analyst, the current program was shown to give more accurate and complete information regarding the actual behavior of the connection.

6 Acknowledgements

The work described herein was supported in part by Hilti North America and Hilti AG. Technical support and advice was provided by Dr. Ulrich Bourgund and Mr. John Silva. The support of these individuals and organizations is gratefully acknowledged. The findings, opinions, and conclusions expressed herein are those of the authors, and do not necessarily reflect those of the sponsoring organizations.

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BIM IN THE ANCHOR INDUSTRY

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ABSTRACT

Building Information Modeling (BIM) has impacted all aspects of construction from the owner to the sub-contractor; from the tower crane supplier to the fastener manufacturer. Historical construction processes for design, analysis, bidding, coordination, scheduling, procurement and fabrication are all transformed in today’s BIM workflows. As the barrier to entry for both BIM enabling hardware and software continues to decline, the pace of adoption for these digital tools on the construction site will continue to quicken as will the disruption of existing practices. This is particularly evident in the non-structural building systems where BIM is rapidly being embraced by trade contractors to facilitate estimation and prefabrication of the systems themselves, as well as the hangers, attachments and supports that go along with them.

This paper gives a brief overview of the impact of BIM on various construction workflows, and specific examples of how BIM will continue to transform the construction fastener industry for non-structural attachments. An example of a holistic BIM design software solution embracing the design of mechanical, electrical and plumbing lines, their support and bracing, as well as the anchoring by means of post-installed and cast-in-place anchors will be presented. The contribution will help to bring BIM to the attention of a broader audience of the engineering and research community.

1 Introduction to BIM

In contrast to 2D drafting which is essentially only a geometrical model representing physical objects, a Building Information Model (BIM) is rich with information for the contents of the model including things like material properties, supplier and ordering information, size, weight, color, installation time, performance data, service intervals and more. Embracing BIM across a construction project can provide significant advantages to all project participants. Prior to construction, owners can conduct a virtual “walk-through” of their future space and work through complex design changes with the architect, and contractors can extract accurate quantity and labor take-offs enable real-time estimating which leads to better decision making on design options. A detailed project schedule and construction sequence can be developed across the various trades, and clashes between building systems can be worked out in advance to optimize the construction schedule. During construction, significant amounts of assembly, fabrication and labor can be performed off-site in a more efficient factory-like environment according to the model. These
prefabricated sub-assemblies of piping systems, structural elements and even entire pre-fabricated rooms can be delivered to the construction site where they are installed and integrated with other building components. Requests for Information (RFIs) and change orders can be tied to actual model elements and resolved efficiently and collaboratively replacing slow paper processes of the past. After construction, a complete as-built 3D model can be turned over from the project team to the owner/operator for future facility management and maintenance. When done correctly, BIM should deliver a higher quality project, at lower costs, in a faster time compared to traditional construction methods.

Figure 1: BIM with mechanical systems highlighted

The promise of BIM is obvious but achieving these benefits in practice has proved to be much more difficult. While the collaborative nature of BIM is one of the main draws and a prerequisite to realizing the benefits of a comprehensive model, this collaboration is at odds with the prevailing construction mentality of each project participant (owners, designers, construction managers, subcontractors) protecting their own profitability. This is proliferated by design-bid-build type construction contracts and existing legal norms in the industry. Some countries such as the U.K. and Singapore have tried to bring BIM to the industry with tops-down mandates and national regulations, however there is evidence that this approach has its limits. The 2017 NBS National BIM Report found that in the UK over half of the respondents (51%) believe that “The Government is not enforcing the Government's 2016 BIM Mandate”

While it may be difficult to force BIM on the construction industry from the top down, it is equally difficult to slow down the adoption of BIM practices when they are embraced from the bottom up.
Modeling programs such as Autodesk Revit have become increasingly user friendly and functional, and in tandem the cost of implementing BIM tools, both software and hardware continues to fall. As discussed, one of the major benefits from a well-constructed BIM is the ability to prefabricate system assemblies, and the mechanical, electrical and plumbing (MEP) trade contractors responsible for the construction of these systems are embracing this BIM-enabled construction methods, driving BIM adoption from the sub-contractor level up through the project teams. In the United States, for example, a recent survey\textsuperscript{2} found that 50\% of the Mechanical Contractors Association of America (MCAA) members now have BIM department compared to 33\% for the general construction industry, and that the mechanical contractor assumed the lead BIM role instead of the general contractor or architect/engineering team the majority of the time. As BIM goes, so does prefabrication with the amount of trade contractor prefabrication work tripling in the US construction market from 13\% in 2010 to 35\% in 2016\textsuperscript{3}. These trends of adopting modeling software and prefabrication will continue and likely quicken.

![Figure 2: Prefabrication shop (Courtesy of Comfort Systems USA)](image)

2 BIM, Prefabrication and Anchors

For typical new construction, the majority of concrete anchors are used for the attachment of non-structural MEP systems such as pipe, duct, conduit, and cable tray. The most common application is anchors used to suspend these systems from the ceiling with hanger assemblies, and the modeling and prefabrication trends in the MEP trades have an acute impact on the selection, design and layout of these anchors.
Historically, the routing of these systems, attachment types and locations were largely decided in real-time at the jobsite. For example, the physical location of a switch gear unit and transformer may have been determined in advance, but the actual electrical conduit connecting the two pieces of equipment, as well as the hangers, anchors and fasteners would be measured, cut and/or installed on the jobsite. When other systems such as duct or sprinkler pipe were in between, the individuals trades would need to work out these potential conflicts, or ‘clashes’ as they are called in the BIM world, in the field with rerouting and rework. Clashes can occur between the systems themselves such as a pipe and duct that need to occupy the same space or between the hangers and anchors used to support these systems which can occur when the space immediately above one system is blocked by another system preventing the installation of a vertical gravity hanger. In seismic regions, suspended systems require additional transverse and longitudinal bracing components to resist potential seismic loads, further occupying the overhead space and increasing the probability of clashes.
When all trade contractors are collaborating to create a federated BIM prior to construction, the clashes can be detected and resolved prior to the work commencing during coordination meetings saving both time and materials on the jobsite.

Figure 5: MEP systems in BIM after clashes have been resolved (left) and actual installation (right)

Clash free models that include all hangers and braces offer significant benefits to the contractor. For example, rather than drilling holes for post-installed concrete anchors in overhead floor decks, preset concrete insert type anchors can be precisely placed prior to the concrete pour using a robotic total station which accurately locates points from a BIM on the jobsite. Switching from post-installed anchors to cast-in-place devices offers many benefits to the contractor including reduced installation time, reduced material costs, reduced inspection costs, eliminating drilling in to post-tension cables or rebar and minimized exposure to concrete silica dust that is otherwise created when holes are drilled in concrete. The efficiencies gained by reducing the number of post-installed anchors alone has been a major driver of BIM adoption with trade contractors, with some industry experts using the reduced number of holes drilled on a particular project as a leading indicator of successful BIM implementation¹.

Figure 6: Installation of post installed anchors (left) and cast in place anchors (right)
The other major efficiency facilitated by BIM is the ability to prefabricate the hanger assemblies in the shop rather than on the jobsite. When these components have been properly modeled, this information can be used to procure and fabricate these assemblies. Hanger and bracing locations can be identified in the model, associated with the anchor points, and these assemblies can be delivered to the jobsite in batches, pre-tagged for where they need to be installed.

![Figure 7: Prefabricated trapeze hangers (left) and clevis hangers (right)](image)

### 3 Problems and Solutions for MEP Anchors in BIM

While the benefits of modeling anchors, hangers and braces are clear, the actual process of creating this content in the model is more challenging. Even a moderately sized project can have tens of thousands of hanger points, and while there is significant redundancy, there is typically hundreds of different hanger configurations across the trades. The complexity is compounded further in seismic zones where transverse and longitudinal bracing is engineered for specific hanger points creating many unique assemblies.

![Figure 8: Longitudinal and transverse seismic bracing on a conduit system](image)
The traditional method of sizing anchors and hangers for MEP systems is to rely on rules of thumb and national standards to determine things like threaded rod size, anchor type and size, strut size, hanger spacing and seismic brace spacing and configuration. Many national trade codes such as SMACNA\textsuperscript{4}, NEC\textsuperscript{5} or NFPA\textsuperscript{8} provide prescriptive tables for sizing and spacing hangers. While these tables and rules generally have enough conservatism built in to handle standard conditions, they are not well suited for the design complexities associated with the variety of hanger configurations, concrete anchor and insert types, and new innovations in the hardware used for these applications. As a result, hangers and anchors are often both over-designed adding undue cost to a project or under-designed creating safety problems and code violations. These problems are amplified by BIM which encourages more systems to be grouped together on common hangers. These configurations create large loads on the hanging assembly, the anchor, and also the structural system (floors and beams) impacting the structural integrity of the building. Properly sizing anchors and strut for these conditions requires complex engineering analysis. Building code requirements for concrete anchor design in particular has become exponentially more complex in the past 15 years, and the nuances of the various failure mode calculation equations are beyond practical use for these national trade standards. Ideally, these code requirements and engineering calculations would be native to the modeling process so the contractor can leverage the benefits of placing anchors and braces in the model, have confidence that these are designed correctly, and communicate the point loads imposed on the structure back to the structural design team.

Figure 9: Multiple conduit runs supported by trapeze assemblies creating point loads > 500 lbs.

Building code requirements for seismic bracing include explicit engineering for nonstructural MEP systems. For example, ASCE7 Chapter 13 “Seismic Design Requirements for Nonstructural Components\textsuperscript{6}” includes specific equations to determine seismic forces on MEP systems and proper detailing requirements for seismic bracing and anchors used to resist these loads. The seismic forces on the components include factors for the ground acceleration ($S_{Dh}$), weight of the system ($W_p$) type of system ($a_p$, $R_p$) importance of the system ($I_p$), and height of the system in the structure ($z$, $h$), Eq. 1. As a result, typical building structures will result in hundreds or even thousands of unique seismic bracing configurations.
Due to the complexity of determining the seismic forces, as well as the aforementioned involvedness of calculating the resistance of components such as strut or anchors, most trade contractors turn over their system layout to specialty engineering contractors for seismic calculations and detailing. Since models are constantly changing and adjusting through the coordination process to resolve clashes with other trades, models are typically not turned over until the later stages of design, at which time there is little opportunity to optimize a seismic bracing system and many difficulties fitting bracing in with existing systems. Furthermore, project time constraints often result in the final bracing designs and calculations being prepared outside of the model on static two-dimension sheets. To fully harness the benefits of BIM, the seismic bracing details, hardware, and calculations should all be included in the BIM during the initial design of the project.

The computational speed and ability of the current BIM programs such as Autodesk Revit can offer some fantastic solutions to automate and optimize the modelling process. When a pipe is modeled in Revit, in addition to an accurate 3D representation of the pipe, a trove of supporting data is added to the model as well such as the pipe type, specification, performance specifications, material, weight, what type of fluid the pipe will be carrying, cost information, labor hours required to install the pipe, supplier information, etc. This information is used to automate the placement of tees, elbows, unions and verify the compatibility with connected system. Once a piping system is completed in Revit, engineering checks on flow, pipe stress and connectivity can be automatically checked.
Following the same logic, the information embedded in the BIM for MEP systems can be used to solve the current problems with anchors, hangers and braces mentioned above. While Revit or other modeling programs do not natively include this functionality, open Application Programming Interfaces (APIs) allow developers to build additional features which can be added-in to these programs. Working with trade contractors, non-structural engineers, industry groups, and experts in the BIM field, DEWALT has built a plug-in for Autodesk Revit to automate and optimize the hanging and bracing of MEP systems called HangerWorks. This world’s first tool uses the data rich content in the model to build hanger and bracing assemblies following engineering principles and code requirements and automatically populate these throughout the model. The weights of the MEP systems, included the contents such as water or wire, are calculated and allocated to each hanger point to determine the gravity and seismic loads required for sizing the anchors as well as the hanger and bracing assemblies.

Assemblies are built with manufacturer specific content, allowing cost optimization of the hanger and bracing system. Calculations and code checks are conducted real-time, and adjusted as the model adjusts so that systems, hangers and braces can be moved through the coordination with continuous engineering feedback for the designer. As these calculations are embedded in the software, a detailed engineering report on any or all hanger and bracing points is readily available, and continuously updated as the model evolves. Most importantly, the major benefits mentioned previously such as resolving clashes in the model, placing concrete inserts prior to the concrete, and prefabricating hanger assemblies are easily achieved when HangerWorks is used to model hanger and bracing components with built-in tools and features for each of these tasks.

![Figure 11: HangerWorks anchor calculations (left) and prefabrication schedules (right)](image)

4 Conclusions and Future Thoughts

The impact of BIM on the construction industry is just beginning to be realized. A detailed building model enables design optimization and streamlining of the construction process to a level that could only be dreamed about just 10 years ago. It is now common practice in some parts of the world to create detailed BIM models for MEP systems to enable coordination with other contractors, complex
prefabrication of systems and hanger assemblies and pre-placement of concrete insert anchors. As the software used to create these models continues to be more predictive and powerful, the ability to automate and optimize this process will continue to progress impacting the entire workflow from the modelling process to the hardware used in the final installation. Tools like DEWALT HangerWorks, a Revit plugin, seamlessly automate major engineering and design processes for anchors, hangers and braces and give a glimpse in to what the future of construction may look like.

References


CONNECTED WITH BIM – A REVIEW OF BIM FOR CONSTRUCTION PRODUCTS LIKE ANCHOR CHANNELS

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ABSTRACT

Building Information Modelling (BIM) is one of the hot topics in the construction industry. The BIM method allows the construction industry to get connected to the economy of knowledge and information technology. BIM offers opportunities also for construction product suppliers as it may be the key for a better and earlier exchange of information along the chain supplier – planner – contractor – owner – occupant. Furthermore, BIM may be used at the construction product suppliers to optimize internal processes. Digitalization of the construction business requires BIM, however, the application of the BIM method is not yet fully standardized and there are still many challenges to cope with.

1 Introduction

Construction projects require a high level of planning for a proper management and coordination of all companies, objects and people involved. This holds in particular for cast-in construction products, e.g. anchor channels (because these cannot be post-installed) and construction products used to connect structural elements, e.g. connection plates (because these link two individual elements which may be in the scope of different companies). The ever increasing complexity and quality of buildings on one side and decreasing available cost and time budgets for construction on the other side call for efficient and preferably digital solutions.

Building Information Modelling (BIM) appears to be the solution and is therefore in the focus of the construction industry. Also construction product suppliers are attracted by BIM as it may be the key for a better exchange of information along the supply chain and storage of the as-built information. Furthermore, BIM allows the connection of all available information as early as possible. Beside these external effects, BIM may also be used internally by the organization of construction product suppliers. In particular customized products, e.g. tailor-made steel elements for the connection of façade components to the reinforced concrete structure, may be designed and produced using BIM to allow a seamless digital and thus efficient process.
2 Background of BIM

BIM is a method to optimize planning, construction and documentation of buildings based on digital information technology. The method was formulated already in 1974 though it was not before 1992 that the three letter acronym was coined. In the 1970s traditional drafting machines (Figure 1a) were the drawing boards in civil and architectural engineering but the increasingly smaller, cheaper and more capable computers blazed the trail of success into the engineering offices with Computer Aided Design (CAD) as one of the core application (Figure 1b). At the beginning, CAD allowed the development of 2D drawings only but soon the design of 3D building models became possible.

![Drawing boards: a) Traditional drafting machine; b) Modern CAD workstation](source of photos: Wikipedia)

The basic idea of BIM was then to use CAD not only as the digital 3D drawing board but to make use of the data to get more out of the system, in particular information of quantities (e.g. area, volume, pieces) and qualities (e.g. materials, climate, functions). Later, the BIM with 3D models was amended by functions to control time (construction schedule, aka BIM 4D) and money (cost estimation, aka BIM 5D). Meanwhile, BIM passed the peak of inflated expectations and the trough of disillusionment typical for innovation hypes. BIM is now steadily gaining recognition among the planners and designers who realized that BIM does not accomplish all the work automatically once the system is set up but that the readily available information provides added value after its systematic and painstaking input. Though the benefits appear to outweigh the risks, however, there are still challenges involved.

The central coordination BIM model is created in the core BIM program, e.g. Revit or ArchiCAD. It is not conventionally drafted but virtually erected with predefined objects like toy blocks. Each object is a member of a BIM object type (aka BIM family) provided in the library of the BIM program. The original library is furnished with predefined BIM object types representing standard building elements, e.g. beams and columns, and may be amended by custom-made BIM object types. These virtual models of components are also referred to as smart objects (Figure 2a) if they are programmed parameterized (e.g. modifiable geometry), with additional information (e.g. load bearing capacity) and with the intelligence to interact with the remaining model of the building (e.g. imperative contact to reinforced concrete elements). At the beginning, the central BIM model is an...
empty shell which is more and more enriched as the planning engineers of the various trades (e.g. architecture, structure and MEP) superpose the data of their models (Figure 2b) which are developed either with the same BIM program or by specialized programs, e.g. AutoCAD, Sofistik and CADduct. The specialized programs do not necessarily have to be object based like the core BIM program.

At the beginning of the second millennium, the countries leading the way, i.e. those of Scandinavia, Netherlands, UK, USA and Singapore, implemented BIM guidelines to be adhered to. Even Germany, which heavily fragmented construction industry with its pronounced surplus capacities lags behind, announced to enforce the BIM method at least for public infrastructure projects after 2020. Codification of BIM is in full swing with the ISO 16739, defining the external interface for data exchange, and the ISO 12006-8-9, dealing with the information organization of construction works, already published. Furthermore, the ISO 29481-10-11 and ISO 19650-12-13 to standardize models, methods, and management were penned.

The standards already provide a good framework how BIM should be implemented at various levels of architecture, engineering and construction (AEC) companies and their teams. Since the codes are still under development, however, isolated approaches at various companies can be observed which foster a Closed BIM approach. In particular an everlasting and independent data exchange format is required to exploit the biggest benefit of the new operation method as Open BIM. Only this kind of BIM culture supports the fair and transparent collaboration and allows holistic analyses e.g. for the certification of the energy and environmental design as a part of a cradle to cradle approach. It is out of question that the multidimensional complexity and information flood of modern construction projects require a complete digitalization which will drive the construction industry to fully adopt BIM. There is just no alternative.
3 BIM in the Construction Industry

The fragmentation of responsibilities at several locations (e.g. engineering offices and construction sites) and over a long period (from conceptual design to final demolition decades if not centuries later) is a major challenge for the AEC industry which is further exacerbated by the following:

- Many parties with many more individual jobs are engaged at construction projects. Therefore, substantial coordination of works and definition of interfaces are required.
- Because virtually every building is unique, automation and standardization of the planning and construction processes is difficult and to date quasi non-existent.
- The uniqueness comes together with the “first time experience” and learning progress only over time, making construction projects often inefficient and troublesome.
- For construction projects, conventional tools and methods with limited digital support do not allow the degree of planning detailing as it is common e.g. in the automotive industry.
- During the complete life cycle of typically 50 to 100 years, the requirements for periodical maintenance checks have to be updated and the records have to be saved.

As a consequence, vast interaction and extensive communication between all parties involved is required where a lot of information may dissipate during the realization of the project and thereafter. Furthermore, the relatively low costs of many items, e.g. embedded parts cast in reinforced concrete elements, do not justify a separate consideration in time bar charts, the traditional tool for scheduling and coordinating the works. For this reason, many works are expected to “self-coordinate” – a method which often fails in particular when time and costs were underestimated during tendering and contracting which frequently leads to understaffed project teams and cuts at the planning jobs.

The fragmentation of responsibilities in space and time typically disconnects the supplier of construction products from the planner in the engineering office and the contractor at the construction site (Figure 3a): The supplier provides product drawings – either for download (typically standardized products) or by email (typically customized products). Next, the planner copies the product drawings into the construction drawings. Finally, the contractor receives the construction drawings. This is the simplified “network” – in reality, the situation is even more complex, e.g. due to interlaced contracts and staggered subcontracts.

The drawing is the only and universal language of engineers and site workers. The drawings are mostly 2D even today and provide very little additional information. The planner may have to call the supplier for additional information he cannot find on the product drawing which reduces his work efficiency. Still, the planner cannot implement all the additional information in the construction drawing and has to concentrate on the most relevant information for the contractor and waive a lot of information (also the information important e.g. for the owner and the occupant). Therefore, the total flow of information is very incomplete and poorly synched with the flow of material. It happens to be that the installer of the contractor is puzzled when the construction product is delivered to the construction site. The information disruption is especially severe between the major phases of projects – i.e. planning phase, construction phase and maintenance phase – because here larger shifts in responsibilities as well as changes of programs have typically to be encountered. In sum, the whole process is not fully streamlined and digitalized with lots of repetitions and losses of work.
The result is often a poor design with an incomplete collection of information scattered in letters, faxes, drawings, emails, messaging apps and personal communications. At the same time, the availability of information anywhere at any time becomes more and more important also in the construction industry\textsuperscript{14}. Recent research has shown that 80\% of the construction defects originate in faulty design\textsuperscript{15}. The resulting cost increase of large-scale projects – for example in Germany the recently completed Hamburg Concert Hall (Elbphilharmonie) + 700 million EUR, the new Berlin Airport (completion originally planned for 2013, opening today uncertain) + 6 billion EUR and the new Stuttgart Railway Station S21 (opening date postponed to 2022) + 8 billion EUR estimated to date – is communicated in the press, however, represent only the tip of the iceberg whereas the real focal point should be put on smaller projects\textsuperscript{16}.

Here comes the BIM method with the underlying concept “first plan the building and then build the plan” into play. BIM ensures a complete flow of information without media discontinuities and enables a cost control and a coordinated flow of material since BIM can also be used for the scheduling and procurement. All involved parties access the same central coordination BIM model which collects all relevant information (Figure 3b): The supplier of construction products develops corresponding smart BIM object types to help the planner designing the building. The BIM objects may be embedded in plug-ins of the BIM program handling the central BIM model or can be downloaded manually and added to the BIM object type library. Currently, several download platforms for BIM object types compete against each other to become the “standard” and thus to achieve a quasi-monopolistic position (who runs the platform earns the money).

The developed BIM model allows the simulation of construction and occupancy of the building and is jam-packed with information at the end of the planning and construction phase. Because all parties involved have access to the BIM model, information can be accumulated and shared on a giving and taking basis. All relevant data is available as early as possible and for those required; e.g. purchasers...
and installers of the construction company may check all objects belonging to a certain BIM object type in the BIM model for specification requirements and installation instructions. While the cost-and time-benefit of BIM for building projects has also been scientifically proven in the meantime, further benefits are yet to be explored, e.g. field to BIM applications or BIM to field applications where survey systems guide the installer to identify and position designated construction products. Whatever development BIM will take, it will be a corner stone for the digitalization of the construction business and thus improve the construction productivity which stagnates at a low level since decades in stark contrasts to all other industries.

4 BIM and Construction Products Today

As pointed out above, the BIM objects are virtual components, e.g. construction products, which have inherent geometric information such as dimensions (length, height, width) and their derivations (surface area and volume), as well as non-geometric information, e.g. material, weight and classifications (static parameters) and lead time, scheduled price and maintenance requirements (dynamic parameters). The parameter names are controlled by the BIM programs and for this reason, are proprietary which hinders the data exchange. The aim of the BIM user community is to standardize the property format on the basis of the Industry Foundation Classes (IFC) requirements. Currently, however, the IFC interface is far from being perfect. Even the simple geometry of a bracket designed with a solid modeling program (Figure 4a) is corrupted when transferred via IFC to a BIM program (Figure 4b). This holds also for the exchange between different core BIM programs. Since the difficulties increase with more sophisticated BIM objects, construction products must be either very simplified or provided in the native format of the BIM program in order to guarantee the functionality of smart objects.

![Figure 4: Representation of JORDAHL® brickwork support bracket JVAeco+](image)

Most important for the coordination, responsibility and reliability of the planners is the level of development (LOD) on the basis of BIM Protocol Form G202 published by the American Institute of Architects which defines LODs from 100 to 500. The BIM Forum publishes annually a refined interpretation of the LODs 100 to 400. The LOD 350 was amended because this intermediate stage is a pivot point for the BIM planning where many components are precisely specified (Table 1). The LOD may be further broken down into the level of (graphical) detail and level of information. The
level of (graphical) detail merely refers to the visual illustration. A perfect optical rendering may be less developed with regard to the stored information and vice versa. Note that aforementioned definitions are neither codified nor unified yet and that the acronym LOD may also be used for the level of (graphical) detail whereas LOI stands for level of information.

Table 1: Definition of the main levels of development (LOD) according to the BIM Forum

<table>
<thead>
<tr>
<th>LOD</th>
<th>Content</th>
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<tbody>
<tr>
<td>100</td>
<td>Components are symbols showing the existence but not its size, shape, location and orientation</td>
</tr>
<tr>
<td>200</td>
<td>Components are generic placeholders and may be volumes for space reservation but still must be considered approximate</td>
</tr>
<tr>
<td>300</td>
<td>Components are exact with regard to quantity, size, shape, location, and orientation (which may require rather specific than generic information of construction products)</td>
</tr>
<tr>
<td>350</td>
<td>Parts necessary for coordination of the element with nearby or attached elements are modeled (which may require rather specific than generic objects representing the construction products)</td>
</tr>
<tr>
<td>400</td>
<td>Components are modelled at sufficient detail and accuracy for fabrication with correct quantity, size, shape, location, and orientation (and make)</td>
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According to the experience of the authors, an indicative and generic graphical representation of connections is sufficient at LOD 200 (Figure 5). The correct quantity, size, shape, location, and orientation of connections, is required in general for LOD 300 and higher, calling for product specific BIM objects of connections. Details such as bolts and nuts, diameter and type are not required below LOD 350.

Figure 5: Anchor channel with channel bolts installed in structural member at the fundamental levels of development (LOD) 100 to 400 according to the BIM Forum

BIM object types must be programmed for the highest LOD for which the objects are intended to be used. Note that different levels of graphical detail (coarse, medium, fine) may be programmed within one BIM object type which can be selected at any time. Typically, graphical details of the BIM objects are turned off when changing to a coarse level of graphical detail. The file size of the BIM object type, however, is the same as when the BIM object is shown with a fine level of graphical detail. This feature only allows the reduction of the computational work load for the static and in particular dynamic display of the BIM model and has no influence on the LOD.
Separate BIM objects may be provided for one and the same construction product to keep the data volume low and at the same time to meet the requirements of separate LODs (Figure 6). However, this approach would require (i) the planner to replace the BIM objects belonging to this particular BIM object type when jumping from one LOD to the next higher LOD and (ii) the supplier to provide several, different BIM object types for one and the same construction product which may be confusing.

Figure 6: BIM object of JORDAHL© anchor channel JTA fulfilling the requirements of a) LOD 350 and b) LOD 400

In conclusion, a codified definition that BIM object representing connections generally should meet the requirements of LOD 400 could be preferable. Consequently, BIM object types for connections would be delivered at the very beginning with all the data and intelligence required during the complete life cycle. The resulting file size of the BIM object type, however, counteracts the BIM code of conduct to reduce the data volume to the max. How to solve this conflict remains to be seen.

The challenging data size for large BIM models is one reason why currently the BIM implementation starts small and why even today the terms of contract often specify LOD 300 as a maximum BIM requirement. At this LOD, only cast-in construction products for structural elements such as anchor channels are required as BIM object. BIM objects representing connections cast in non-structural elements, e.g. support corbels (Figure 7a), or objects representing post-installed connections, e.g. connection plates (Figure 7b), may not be needed before LOD 350 or higher. However, the BIM implementation is expected to be scaled fast which will push the demand of BIM object types representing construction products including any type of connection.
A good example for a smart BIM object is the JORDAHL® anchor channel JTA with channel bolts (Figure 8a). This anchor channel is embedded together with matching channel bolts within the same BIM object type. Each BIM object type contains different profile sizes in staggered standard lengths of 100 mm to 6,000 mm, offering a variety of more than 100 anchor channel sizes. In addition, the material can be selected for the anchor channel and channel bolts (galvanized and stainless steel) in various strength classes, thread sizes and bearing lengths.

The BIM object type for JORDAHL® anchor channels JTA with channel bolts allows not only to make use of the standard advantages of BIM, e.g. clash detection and quantity survey. In addition, the intelligence is programmed that the objects require the embedment in concrete and the modification of many parameters, e.g. type, length and positioning of anchors, channel and channel bolts is possible. For example, the position of the anchors may be shifted along the channel according to the requirements, e.g. in order to accommodate the reinforcement during construction, and the position of the channel bolts may be changed, e.g. in order to allow the fixation of new components due to a refurbishment.
5 BIM and Construction Products in the Future

The concept of a fully digitalized process starting from the customer’s design and selection of the product, quoting, ordering, manufacturing, shipping and billing is known in Germany as Industry 4.0\textsuperscript{23} – a succinct term which becomes also internationally popular and proclaims the 4th disruption in industrial production. Though a universal disruption in industrial production has not be proven yet, the current changes may have a resounding impact and the crucial transition phase may be short. In contrast to the earlier disruptive changes in the industrial history, the production following the 4th disruption is expected to have still large customization numbers but small lot sizes (Figure 9). In other words, not mass off-the-shelf construction products but mass customized construction products may be the future focus.

![Figure 9: The four disruptive changes in the industry in relation to lot size and customization number](source of photos: Wikipedia, Volkswagen, UL)

BIM can be a tool of the construction product suppliers for the realization of Industry 4.0 because BIM makes an individual, single-unit production affordable and competitive due the digital communication along the complete supply chain: The work flow starts with the customized design of the construction product according to the order of the customer or the requirement of the structural design carried out directly in the core BIM program and/or with interchanging specialized programs\textsuperscript{24}. Following, the quantity takeoff of the construction products is transferred from the BIM program to the general Enterprise Resource Planning (ERP) program of the producer. The ERP program is then used to manage everything from material planning and production control to documentation and commissioning. In the end, the customer receives a unique and identifiable construction product.
A fully digitalized process also allows customized construction products to become part of the Internet of Things. Typical applications are tracking, routing (dispatch – installation) and tracing, e.g. by means of code labels, as well as positioning, e.g. on the basis of RFID technology. Further and more active applications, such as cyber-physical systems, currently appear to be fiction for construction products, however, the future will tell – the meaning of integrated circuits was also underestimated at the dawn of the electronic age.

6 Summary

The advantages of the Building Information Modeling (BIM) for the construction industry are compelling: A proper and comprehensive planning of buildings with the BIM method allows the development of realistic virtual models which support the engineering and eventually the construction and maintenance of projects. This approach may significantly reduce the required time and money and increase the quality at any stage of the projects. BIM is the connection of objects and information and therefore will certainly become an important part of the digitalization, in particular for construction product suppliers of customized products, e.g. used for steel-to-concrete connections.

The views expressed in this paper are the views of the authors only and do not necessarily reflect the views of Jordahl GmbH.

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MATERIALS
ABSTRACT

The principles of sustainable development and green building have made strong impact on construction industry at an accelerating rate in recent years. Nano technology is one of the most promising areas of science. The use of nano materials in concrete is a new revolution. Nano materials like nano Silica (nS), carbon nano tubes, nano alumina etc. are presently used in concrete to modify its strength properties. In the present study strength properties such as compressive strength and split tensile strength of M20 grade of concrete with use of Silica Fume (SF) (4%, 6%, 8%, and 10%), and nS (2%, 4%, and 6%) and combination of nS and SF as partial replacement to OPC have been studied. The compressive and split tensile strengths have shown gradual increase for mixes with SF up to 10%. For mixes with more than 2% of nS have not shown increase in strength. The combination of 2% nS and 8% SF have given optimum strengths. Superior strengths of mixes can be achieved with incorporation of SF, nS and right combination of SF and nS.

1 Introduction

Concrete is the most consumed construction material only next to water. Undoubtedly cement is the costliest ingredient of concrete. Cement clinkers production consumes large amounts of energy, associated release of CO₂ emissions has a notable environmental impact. For instance, 1.7 tonnes raw materials is needed to produce 1 ton clinker which leads to emission of 850 kg of CO₂ is per ton of clinker produced. Therefore, reducing the amount of cement to the extent possible without reducing the performance of concrete is going to be very significant for large projects in construction sector. In this context, the use of supplementary cementations materials have a vital role to play in concrete production.

The reaction between the cement and water yields Calcium Silicate Hydrate (C-S-H), which gives concrete strength and other mechanical properties of concrete, as well as some by-products including Calcium Hydroxide (CH), ‘gel pores’ etc. The major issue in concrete at the fresh and hardened state is the crack and its consequent problems, the cracks in concrete structures and premature erosion are mainly due to alkali silica reaction, which is a chemical reaction that causes fissures in the concrete. Apart from the above, permeability of gases through pores and micro-cracks in the concrete, which leads to corrosion problem in the reinforcement of concrete causes further deterioration. Moreover, the expansion and shrinkage in concrete, which are also the causes for crack in concrete at later ages, are mainly due to the sulphate attack, which causes disintegration in
concrete, chemical leaching and both the events are mainly due to the excess Calcium Hydroxide, the by-product during cement hydration.

The supplementary cementitious materials such as pulverized fly ash, ground granulated blast furnace slag, condensed SF, rice husk ash, metakaolin etc. have been studied extensively in concrete as pozzolanic materials to enact the CH and get the additional C-S-H. The addition of supplementary cementitious materials in concrete will not only improve the mechanical properties of concrete, but also its workability, alteration in setting times and durability.

The nano science and nano-engineering (nano-modification) of concrete are terms that have come into common usage and describe two main approaches of applications of nanotechnology in concrete. Until today, concrete has primarily been seen as a structural material. The researchers are now capitalizing on nanotechnology to innovate a new generation of concrete materials that overcome cracking and permeability problems in concrete and are trying to achieve the sustainable concrete structures. Evolution of materials is need of the day for improved or better performance for special engineering applications and modifying the bulk state of materials in terms of composition or microstructure or nanostructure has been the established route for synthesizing new materials. The newer materials can also be obtained by intelligent and intermixing of existing materials at element level. With the advancement of nano technology, nano materials have been developed that can be applied to concrete mix designs to study the physical, chemical and enhanced mechanical properties of concrete. Among the various developed or manufactured nano materials such as nS, nano Alumina, nano Titania, nano Zirconia, nano Fe₂O₃ etc. Carbon nano tubes (CnT) or wires etc. The addition of nS enhances the possibility for the reaction with Calcium Hydroxide (CH) to develop more strength carrying structure of cement Calcium Silicate Hydrate (C-S-H) and also pore filling effect of nS in the concrete. Use of nS in HPC and SCC improves the cohesiveness between the particles of concrete and reduces segregation and bleeding. Concretes with strengths as high as 100 MPa with high workability, anti-bleeding properties and short demoulding time can be produced. nS can be used as an additive to eco concrete mixtures. In the case of eco concrete mixtures, industrial wastes such as fly ash, blast furnace slag are used as admixtures at certain percentages as replacement to cement. Certain problems like longer setting time, lower compressive strength at higher percentages can be overcome by adding nS which improves these properties.

Silica is the common name for materials composed of silicon dioxide (SiO₂) and occurs in crystalline and amorphous forms. SF or micro-silica (SF) is a byproduct of the smelting process in the silicon and ferrosilicon industry. It is a grey colored powder, similar to Portland cement or fly ashes. It is an ultrafine powder collected as a by-product of the silicon and ferrosilicon alloy production and consists of spherical particles with 95% of particles less than 1 micro meter. Specific gravity being 2.2. The main field of application is as pozzolanic material for high performance concrete.

Nano Silica is typically a highly effective pozzolanic material. It normally consists of very fine vitreous particles approximately 1000 times smaller than the average cement particles. Particle size of 5-40 nm, specific gravity being 1.30 to 1.32. It has proven to be an excellent admixture for cement to improve strength and durability and decrease permeability. nS reduces the setting time and increases the strength (compressive, tensile) of resulting cement in relation with other silica components. nS is obtained by direct synthesis of silica sol or by crystallization of nano-sized
crystals of quartz. Moreover, it has been accepted through experiments that, nS particles not only are environment-friendly but also could lead to better results in comparison with SF. nS blended concrete’s long term characteristics can take care of the environment, the consumed concrete and the workers’ health. nS can improve the concrete properties and provides concrete with higher durability. In addition, it helps to save the cement, improve the water and chemical impermeability, decreases the final cost of work, and lower environmental contaminations.

Mazloom et al. (2004) carried out the experimental work on short and long-term mechanical properties of high-strength concrete containing different levels of SF. Their study results indicated that as the proportion of SF increased, the workability of concrete decreased but its short-term mechanical properties such as 28-day compressive strength and secant modulus improved. The compressive strength of concrete mixtures containing SF did not increase after the age of 90 days. Bhanja and Sengupta (2002), carried out extensive experimentation to determine the isolated effect of SF on the tensile strength of concrete at water to binder ratios ranging from 0.26 to 0.42 and cement replacements of 0% to 30%. The compressive, as well as the tensile, strengths increased with SF incorporation. The optimum SF replacement percentages for tensile strengths have been found to be a function of w/c ratio of the mix. The optimum 28 day split tensile strength has been obtained in the range of 5% to 10% SF replacement level, whereas the value for flexural strength ranged from 15% to 25%. Increase in split tensile strength beyond 15% SF replacement is almost insignificant but considerable increase in flexural tensile strength have occurred even up to 25% replacements. Siddique (2011) studied the effect of SF on the workability, porosity, compressive strength, splitting tensile strength, flexural strength, creep and shrinkage of concrete. And he found that addition of SF increases the 28 day compressive strength. SF does not have significant influence on the splitting tensile strength of concrete and there was a steady increase in the flexural strength with increase in the SF replacement percentage.

Ji (2005) experimentally studied the water permeability resistant behavior and microstructure of concrete with nS. nS can absorb the Ca(OH)$_2$ crystals, and reduce the size and amount of the Ca(OH)$_2$ crystals, thus making the interfacial transition zone (ITZ) of aggregates and binding paste matrix denser. The nS particles can fill the voids of the C-S-H gel structure and act as nucleus to tightly bond with C-S-H gel particles, making binding paste matrix denser, so long-term mechanical properties and durability of concrete are expected to be increased. Quercia and Brouwers (2010) carried out studies on nS as a high potential concrete additive to reduce the cement content in concrete mixes. Givi et al. (2010), investigated the size effects of SiO$_2$ nano-particles on compressive, flexural and tensile strength of binary blended concrete. SiO$_2$ nano-particles with two different sizes of 15 and 80 nm have been used as a partial cement replacement by 0.5%, 1.0%, 1.5%, and 2.0% by wieght. The results show that the SiO$_2$ nano-particles blended concrete has higher compressive, flexural and tensile strength at all ages of moist curing in comparison to concrete without nS. Khanzadi et al. (2010), have carried out studies on influence of nS particles on the mechanical properties and durability of concrete. They added 1.5%, 3% nS. Compressive and tensile strength of the concrete increase with replacing the nS, especially at early ages. Their results indicated that the pozzolanic activity of nS is higher. nS consumes calcium hydroxide crystals, reduces the size of the crystals at the interface zone and transmute the calcium hydroxide feeble crystals to the C-S-H crystals, and improves the interface zone and cement paste structures. Zhang
and Islam\textsuperscript{8} (2012), conducted experimental study to evaluate the effects of nS on rate of cement hydration, setting time and strength development of concretes with about 50\% fly ash or slag. They found that length of dormant period was shortened, and rate of cement and slag hydration was accelerated with the addition of 1\% nS in the cement pastes with high volumes of fly ash or slag, and the incorporation of 2\% nS by mass of cementitious materials reduced initial and final setting times by 90 and 100 min. They increased 3 and 7 day compressive strengths of high-volume fly ash concrete by 25\% and 30\%, respectively, when compared to the reference concrete with 50\% fly ash.

Qing\textsuperscript{9} et al. (2007), have studied the influence of nS addition on properties of hardened cement paste (hcp) as compared with SF and they studied through measurement of compressive and bond strengths of hpc. nS made cement paste thicker and nS accelerated the cement hydration process. Compressive strengths of hcp and bond strengths of paste-aggregate interface using nS were higher than those incorporating SF. The pozzolanic activity of nS is much greater than that of SF. Tobón\textsuperscript{10} et al. (2010), studied some physical properties of Portland cement replaced with nS in percentages of 1\%, 3\%, 5\%, and 10\%. They showed that the nS from 5\% to have a major positive influence on the mechanical strength of mortars. Nili\textsuperscript{11} et al. (2010), they used different content of microsilica and colloidal nS as partial replacement of cement in the concrete mixture with 0.45 water-cement ratio. Cement replaced by SF (1\%, 1.5\%, 3\%, 5\%, and 7.5\%) and colloidal nS (1.5\% and 3\%). They have attained the highest compressive strength at the ages of 7 and 28 days when the mixtures contained 6\% SF and 1.5\% nS. Tavakoli and Heidari\textsuperscript{12} (2013), studied the effect of simultaneous use of nS and SF in concrete. The SF was used in quantities of 5\% and 10\% and nS was 0.5\% and 1\% of the cement. Using both 10\% SF and 1\% nS, as a cement replacement, resulted in 42.2\% increase in compressive strength in comparison to control sample. Gupta\textsuperscript{13} (2013), carried out experiments using nS in cement mortar with SF (5\%, 10\%, and 15\%) alone and nS (3\%, 6\%, 10\%, and 12\%) alone and it was reported that with 5\% replacement of cement by nS (mean size 15±5 nm), 7 and 28 days compressive strength of mortars were increased by 20\% and 17\%, respectively, whereas 15\% SF replacement increased mortar strengths by 7\% and 10\% compared with those of control Portland cement mortar. Babu\textsuperscript{14} (2013), carried out experiments on the properties of blended cement with nS and SF. The test specimens were cast with 5\% to 9 \% SF and 1\%, 2\%, 3\%, and 4\% of nS particles for test specimens. Results indicated that, setting times were increased with increase in percentage of nS in cement blended with SF. A combination of 6\% SF + 3\% nS was given the best performance in compressive strength. Wahab\textsuperscript{15} et al. (2013), studied mechanical properties of a combination of mineral admixtures like nS and condensed SF with OPC is used as replacement to cement in certain proportions on high strength concrete. Hussain and Shastry\textsuperscript{16} (2014), studied the strength properties of concrete by using micro Silica and nS, and found out the strength properties gets magnified significantly. The present work aims to study the mechanical performance of concrete with incorporation of SF, nS and combination of SF and nS.

2. Materials and Methods

Cement used for the test procedure was 43 Grade Ordinary Portland cement. The river sand used has a specific gravity of 2.63 and belonged to zone I. Crushed granite stone used as coarse aggregate, graded passing 20 mm and retained on 12.5 mm were used.
Silica fume is an ultra-fine powder, with particle sizes less than 10µm and is light to dark grey in color. It is a co-product from the silicon or ferrosilicon industry and is rich in silicon-di-oxide (SiO₂). Because of its pozzolanic and void-filling properties the addition of SF to concrete provides ultra-high compressive strengths.

Silica fume brand used for the experiment is CORNICHE SF and it complies with the following standards: ASTM C 1240 (2005) and IS 15388 (2003) Standards.

Nano Silica particles when added to concrete make it easier to pump and present no segregation problems like micro silica. They also help in reducing the amount of binder in each concrete mix and give better results. Nano fillers enter micro and Nano pores and give denser concrete which has a high resistance to sulfate and chloride attacks, is less permeable and of course has a higher resistance to compaction. Better packing of the C-S-H leads to a high density C-S-H which is practically a guarantee for slowing creep thus multiplying various times the lifespan of the concrete structure.

### 2.1 Methodology

Table 1, presents the test matrix of experiments undertaken. The nominal mix proportion used is 1:1.5:3. Water cement ratio of 0.50 was chosen. The concrete was mixed in a concrete mixer and poured into moulds of size 150 mm × 150 mm × 150 mm cubes and cylinders of size 150 mm diameter × 300 mm height with sufficient vibration over the vibrating table. The cubes and cylinders were demoulded after 24 hours and cured for 28 days in water. Cubes and cylinders were taken out of the curing tank after 28 days and kept outside to dry till the experiments were carried out.

Table 1: Test matrix for Compressive and Split tensile strengths

<table>
<thead>
<tr>
<th>nS</th>
<th>SF</th>
<th>No of cubes</th>
<th>No of cylinders</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>0</td>
<td>4</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>0</td>
<td>6</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>0</td>
<td>8</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>0</td>
<td>10</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>3</td>
<td>3</td>
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<tr>
<td>4</td>
<td>6</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>4</td>
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</tr>
</tbody>
</table>

The w/c ratio was taken as 0.5. Mixes were replaced in 2%, 4%, and 6% by weight of cement, for nS, 4%, 6%, 8% and 10% for SF and (2%, and 8%), (4%, and 6%) and (6% and 4%) for combinations of nS and SF respectively.
3 Results and Discussion

3.1 Slump Tests
Initial experiments were conducted to know the workability variation without super plasticizers on concrete incorporating SF, nS and both (SF & nS). Standard slump cone apparatus is used for measuring the value of slump. During the slump tests, it was found that the workability of fresh concrete made with 6% replacement of nS with cement had slump value close to 25 mm. With increase in percentage of nS in concrete the workability was getting lower and lower.

Workability test of fresh concrete made with 10% replacement silica Fume also had the lower slump value nearer to 10 mm, and when both nS and SF are used to replace up to 10% of cement again the slump value was getting reduced. The experiments were conducted to achieve a slump in the range of 70-80 mm with appropriate dosage of super plasticizer (Conplast SP 430) for all mixes. The results of slump tests and plasticizer dosages for fresh concrete incorporating nS, SF and both are presented in Figure 1.

![Figure 1: Slump variation with nS, SF, and both (nS & SF) addition](image)

It is seen that a remarkable reduction in the workability of the concrete mixes, due to the addition of nS, SF and both of them to the concrete. It is because of the instantaneous interactions between nS and SF and the liquid phase of the cementitious mixes (mainly dissolved alkalis) that cause the formation of gels which are characterized by high water retention capacities, making the concrete less workable. However, to achieve the slump in the range 70-80 mm, super plasticizers were added in adequate dosages.

3.2 Compressive Strength
Compressive strength testing of all specimen were carried out as per IS 516 (1959). The load was applied without shock at a rate of 140 kg/cm²/min. A set of three cubes were tested for all eleven mixes, for each percentage of replacement. The maximum load resisted divided by cross sectional area of specimen, gives the compressive strength. Average of three specimen were taken, provided the individual variation in strength was not more than ±15% of the average, and the results were tabulated and interpreted.
3.2.1 Mixes Incorporating nano Silica

Figure 2 presents the strength results for concrete with nS. It can be noticed that the compressive strength of concrete suddenly increased to 44.7 MPa when 2% of OPC is replaced by 2% nS. And then, it goes on decreasing with increase in percentage of nano Silica upto 6% for which the strength is 40.1 MPa. This is due to complete utilization of calcium hydroxide by 2% nano-Silica. For any amount more than 2%, there shall be no activation. So for 4% and 6% replacement with OPC by nS, it only acts as a filler material and hence strength decreases.

![Figure 2: Variation of compressive strength for mixes incorporating nS](image)

From the Fig. 2, it is seen that the trend line of variation of compressive strength between 2% to 6% is, \( f_{ck} = -2.9767 \times (\% \text{nS}) + 50 \)

3.2.2 Mixes Incorporating Silica Fume

Figure 3 presents strength variation with SF. With the increase in percentage of silica fume the compressive strength was observed to increase. After 28 days of curing, the compressive strength got increased by 3.9% for 4%, and 18.8% for 10% SF replacement. This is due to pozzolanic reaction of silica fume that results in a reduction of the amount of calcium hydroxide in concrete and formation of C-S-H. It is responsible for an additional increase in strength and chemical resistance and decrease in water absorption.

![Figure 3: Variation of compressive strength for mixes incorporating SF](image)

From the Fig. 3, it is seen that the trend line for compressive strength variation with SF between 4 to 10% is, \( f_{ck} = 1.6945 \times (\% \text{SF}) + 36.5 \).
3.2.3 Mixes Incorporating Silica Fume and nano Silica

A total of 10% of cement is replaced with the combination of nS and SF. For mix one (2% nS and 8% SF) the compressive strength, got increased by 21% when compared to control mix. With the increase in percentage of nS in the combination of nS and SF, the strength was found to be decrease but still greater than the strength of control mix. A graphical view of the results is shown in Figure 4. From the figure 4 it is seen that the trend line of variation of compressive strength between 8% to 4% SF, and 2% to 6% nS is, $f_{ck} = -2.6133 \times \text{Mix (nS + SF)} + 39.9$.

![Figure 4: Variation of compressive strength for mixes incorporating SF and nS](image)

Note: (Mix 1 = 0% nS + 0% SF, Mix 2 = 8% nS + 2% SF, Mix 3 = 4% nS + 6% SF, Mix 4 = 2% nS + 8% SF)

3.3 Split Tensile Strength

3.3.1 Mixes Incorporating nano Silica

Figure 5, presents strength variation of concrete with nano Silica. An increase of 15.9% strength is observed. An increase of 3.5 MPa for 2% nS and then for further increase in nano Silica replacement, the strength gets reduced. For 6% nS replaced concrete mix, strength was reduced to 3.1 MPa.

![Figure 5: Variation of split tensile strength for mixes incorporating nS](image)

From Fig. 5, it is seen that the trend line of variation of split tensile strength with nS (2 to 6%) is, $f_t = -0.2567 \times (\% \text{nS}) + 4$.
3.3.2 Mixes Incorporating Silica Fume

Figure 6, presents strength variation with Silica Fume. With increase in percentage of Silica Fume, the split tensile strength was found to gradually increase. The split tensile strength got increased by 5% for 4% replacement of OPC by Silica Fume and increased to 11% for 10% SF.

Figure 6: Variation of split tensile strength for mixes incorporating SF

From Fig. 6 it is seen that the trend line of variation of split tensile strength between 4% to 10% SF is, \( f_t = 1.6945 \times (\% \ SF) + 36.5 \).

3.3.3 Mixes Incorporating Silica Fume and nano Silica

For mix one (2% nS and 8% SF) split tensile strength got increased by 11% when compared to control mix. With the increase in percentage of nS in the combination of nS and SF, the split tensile strength was found to decrease and but still the strength was greater than that of control mix. The results are shown in Fig. 7. From the Fig. 7, it is seen that the trend line for variation of split tensile strength between 8% to 4% SF, and 2% to 6% of nS is, \( f_t = -2.6133 \times [\text{Mix (nS + SF)}] + 50.5 \)

3.4 Relationship Between Compressive Strength and Split Tensile Strength

3.4.1 Mixes Incorporating nano Silica

Graphical comparison of results between compressive strength and split tensile strength of mixes incorporating nS is shown in Fig. 8. From Fig. 8 it is found that the split tensile strength is linearly increasing with compressive strength for mixes incorporating nano Silica, and the equation for trend line of variation is, \( f_{ck} = 0.0759 \times f_t + 0.1201 \).

Figure 7: Variation of split tensile strength for mixes incorporating SF and nS
3.4.2 Mixes Incorporating Silica Fume

Graphical comparison of results between compressive strength and split tensile strength of mixes incorporating SF, are shown in Fig. 9.

\[
y = 0.0486x + 1.2351
\]

From Fig. 9, it is found that the split tensile strength is linearly increasing with compressive strength for mixes incorporating Silica Fume, and the equation for trend line of variation is, \( f'_{\text{sk}} = 0.0486 \times f'_{\text{c}} + 1.2351 \).

3.4.3 Mixes Incorporating Silica Fume and nano Silica

Graphical comparison of results between compressive strengths and split tensile strengths of mixes incorporating nS and SF are shown in Fig. 10.
From Fig. 10, it is found that the split tensile strength is linearly increasing with compressive strength for mixes incorporating Silica Fume and nano Silica, and the equation for trend line of variation is, $f_{ck} = 0.0388 \times f_i + 1.552$.

4. Conclusions

1. Workability of cement concrete got decreased by increase in nS and SF. This warrants use of super plasticizers to achieve same degree of workability as that of concrete with OPC only.
2. Compressive and splitting tensile strengths were observed to increase with increase in SF replacing OPC up to 10%. Nearly 20% and 13% increase in compressive and splitting tensile strengths were observed respectively.
3. Increase in compressive strength by 18% and by 16% in split tensile strengths were observed for replacement of OPC by nS. The strength decreases with increased percentage of nS above 2%.
4. Cement replacement up to 8% with SF and up to 2% with nS, has given high compressive and splitting tensile strengths, compared to other combinations dealt.
5. Compressive strength and splitting tensile strengths predicted equations have been proposed for SF based, nS based, and SF and nS based concretes.
6. Predicted equations for splitting tensile strength from compressive strengths are also proposed for the above three concretes studied.

References:


INVERSE MODEL FOR PULLOUT DETERMINATION OF STEEL FIBERS

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ABSTRACT

Fiber reinforced concrete (FRC) is a material with increasing application in civil engineering. Here it is assumed that material consists of a great number of rather small fibers embedded the concrete matrix. It would be advantageous to predict mechanical properties of FRC using nondestructive testing; unfortunately, many testing methods for concrete are not applicable to FRC. In addition, design methods for FRC are either inaccurate or complicated. In three-point bending tests of FRC prisms, it has been observed that fiber reinforcement does not break but simply pulls out during specimen failure. Following that observation, this work is based on an assumption that the main components of a simple and rather accurate FRC model are mechanical properties of the concrete matrix and fiber pullout force. Properties of a concrete matrix could be determined from measurements on samples taken during concrete production and fiber pullout force could be measured on samples with individual fibers embedded into concrete. However, there is no clear relationship between measurements on individual samples of concrete matrix with a single fiber and properties of the produced FRC. This work presents an inverse model for FRC that establishes a relation between parameters measured on individual material samples and properties of a structure made of the composite material. However, deterministic relationship is clearly not possible since only a single beam specimen of 60 cm could easily contain over 100000 fibers. Our inverse model assumes that probability density function of individual fiber properties is known and that global sample load – displacement curve is obtained from the experiment. Thus, each fiber is stochastically characterized and accordingly parameterized. A relationship between fiber parameters and global load – displacement response, the so-called forward model, is established. From the forward model, based on Levenberg – Marquardt procedure, the inverse model is formulated and successfully applied.

1 Introduction

Fiber reinforced concrete consists of a large number of fibers embedded into concrete matrix. Fibers are distributed within the matrix stochastically, e.g., see Sampson¹ but probability distribution functions are assumed to be known, or could be determined from some experiments or scans. Knowledge of the probability distribution function enables computer generation of material samples
needed for numerical experiments. Chapter 2 describes the distribution of fibers and their orientations within the material. In addition, we compare generated distributions with distributions from experiments that we have performed, see X-ray scans in Kalinčević².

Large number of fibers is the main obstacle in the development of material model. Among many approaches to address the problem one usually applies conforming finite element model where fibers are discrete and are placed along finite element edges, see Smolčić and Ožbolt³. Conforming approach results with huge models that are very demanding in computer resources. However, any computer model requires parameters that are usually difficult to obtain. Ad hoc approach consists in numerical experimentation with parameter analysis where material parameters with most convenient results are chosen as model parameters. Such an approach has many limitations, like difficulty to generalize on the results because parameter determination problem is not formulated as an optimization problem. The most severe consequence is that there is no clear view of the relation between local and global extremes in the parameter space.

Forward model presented in this work is developed only to the extent needed for the proper inverse model formulation and parameter determination. Our forward model presents steps leading from stochastic material characterization to generation of the specimen load – displacement curve. In Chapter 3 two most common probability distributions are presented as applied to two possible choices of parameters. Although the model takes into account only fibers without the matrix, there are no conceptual limitations in upgrading the model so that a concrete matrix model is taken into account, e.g., see Ožbolt and Ananiev⁴. Forward model can be based on Monte Carlo simulation but for successful formulation of the corresponding inverse model it had to be reformulated based on ‘order statistics’, see Rinne⁵.

Inverse model is presented in Chapter 4; it is based on the Levenberg – Marquardt procedure, see Kožar et al.⁶. Formulation of the inverse model is made possible by applying the order statistics that produces unique results for given parameters of a probability distribution function and alleviates the need for Monte Carlo simulations. The accuracy is controlled with proper choice of subdivisions (histogram ‘bins’) in the domain of the probability distribution function. Number of measurement (control) points on the load – displacement curve is arbitrary and does not significantly affect the performance of the inverse model.

Application of the Levenberg – Marquardt procedure requires calculation of sensitivity parameters so sensitivity analysis is also performed and briefly presented in Chapter 5. There are also examples of the inverse procedure and results of parameter estimation. Fiber material parameters estimated from sample load – displacement curves are very accurate. However, some simplifications should be mentioned. In our inverse model, at the current stage of the research, we assume only steel fibers and disregard the influence of the concrete matrix in inverse parameter determination. In addition, only the simplest tensile experiments and corresponding load – displacement curves are addressed. Considering the mentioned limitations, the current inverse model can be thought of as a proof of concept. Nevertheless, there is no inherent limitation that would prevent the inverse model from being extended to include a model of the concrete matrix.

Throughout the paper all lengths have unit of [m], material modulus [kN/m²] and forces [kN].
2 Material Stochastic Properties

Distribution of fibers within the concrete matrix is the main parameter of fiber reinforced concrete. One can take arrangement of fibers from X-ray scans or some other imaging procedure but in order to establish a relation between fiber distribution and loading response, it is necessary to parameterize the fiber distribution. This stochastic parameter is characterized with its probability distribution function. Furthermore, knowledge of the probability distribution function enables computer generation of synthetic material samples suitable for numerical experimentation.

Generation of fibers within the matrix starts with generation of points in x and y directions (for 2D models that are the only ones addressed in this work). Points are generated independently according to the uniform distribution \( U(x; a, b) = 1/(b - a) \), where \( a \) and \( b \) represent the interval of the distribution. However, points are not uniformly distributed within the specimen but are clustered according to the Poisson distribution \([\text{Sampson 2009}]\) \( P(X) = \frac{\lambda^k}{k!} e^{-\lambda} \). Poisson distribution gives the probability of a fiber being within a sample segment.

After points are generated, fibers with orientation \( \theta \) are introduced at their positions. Fibers length is assumed to be known and orientation \( \theta \) is generated stochastically according to the uniform distribution within the range \([0,2\pi]\). Mathematical model that relates material properties and load–displacement diagram requires knowledge of fiber length or fiber stiffness. They are completely determined with the probability distribution function (that is different from the pdf of their orientation \( \theta \)). For fibers with orientation \( \theta \) distributed uniformly within \([0,2\pi]\), it is possible to calculate the probability distribution of fiber length or fiber stiffness. The length and stiffness distribution follows \( x = \cos \theta \) and cosine distribution belongs to the family of ‘location-scale distributions’; it is calculated according to the rule for transformation of random variables \( pdf(x) = pdf(\theta) \left| \frac{d\theta}{dx} \right| \). The result is the cosine probability distribution

\[
pdf(x) = \begin{cases} 
\frac{1}{\pi \sqrt{1-x^2}}; & -1 < x < 1 \\
0 & \text{otherwise} 
\end{cases}
\]

(1)

Numerical experiments require generated material samples that are discretized with finite elements and analyzed for different loading conditions. Synthetic material samples are generated in Mathematica\(^7\) according to a given probability distribution function; comparison with X-ray is presented in Figure 1.

![Figure 1: a) X-ray scan of a beam sample 4x4x16 cm, b) generated fibers](image-url)
It is visible from Figure 1 that fiber generation is quite realistic and can be used for numerical simulations. This result is confirmed in numerical experiments with Monte Carlo simulation.

3 Forward Stochastic Model

The main purpose of the forward model is to provide parameterization that relates material parameters and experimental results and is suitable for the formulation of an inverse model. Forward model can be easily built using the Monte Carlo method but such a model is not suitable for inverse model formulation. Instead, forward (and later inverse) model is based on order statistics. Order statistics is characterized with iid (independent identically distributed) variables \( X_i \) being arranged in ascending order \( X_{1:n} \leq X_{2:n} \leq \cdots \leq X_{n:n} \).

Transition from ordinary to order statistics is essential for success of the inverse procedure. We will explain order statistics on the normal distribution model with \( pdf(x) = \frac{1}{\sigma \sqrt{2\pi}} \exp \left[ \frac{-1}{2\sigma^2} (x - \mu)^2 \right] \).

Monte Carlo simulation with mean \( \mu = 1.0 \) and standard deviation \( \sigma = 0.25 \) for length of 500 bars gives results as presented in Figure 2.a (each simulation produces somewhat different results).

![Figure 2: a) 500 normally distributed bar lengths, b) sorted bar lengths and their bin representatives](image)

In Figure 2.b there are sorted values of randomly generated bar lengths. Simulation domain is divided into segments - bins and a histogram is produced, see Figure 3.

![Figure 3: a) normalized histogram of 50 bins for 500 normally distributed bar lengths generated in Monte Carlo experiment, b) normalized histogram with bin representatives from order statistics pdf](image)
In Figure 3 normalized histograms for 50 bins are presented, i.e., bin width is adjusted so that $\sum_{\text{bins}} h_{\text{bin}} = N_{\text{simulations}} = 500 \,(\text{in this example})$. In Figure 3.a, the normalized histogram is from Monte Carlo simulation and in Figure 3.b from bin representatives for order statistics. Figures 2 and 3 support the intuitive feeling that sorting of the randomly generated values does not change its statistics; mathematical proof will not be given here. In our models, we will use order statistics and bin representative values generated from the probability distribution function and not from the Monte Carlo simulation. However, Monte Carlo will be used for generation of load - displacement curves as a substitute for real experiments.

### 3.1 Stiffness vs. Length Stochastic Model

Forward model for load – displacement curve is formulated as a sum of forces from individual bar contributions, which can be obtained with either bar stiffness or bar length as random parameters:

$$F_{\text{bar}} = \begin{cases} \frac{EA_{\mu}}{L_{\text{stoch}}} \delta & \text{if } \frac{\delta}{L} \leq d_t \\ 0 & \text{otherwise} \end{cases}$$

$$F_{\text{bar}} = \begin{cases} \frac{EA_{\text{stoch}}}{L_{\mu}} \delta & \text{if } \frac{EA_{\text{stoch}}}{L} \delta \leq F_t \\ 0 & \text{otherwise} \end{cases} \quad (2)$$

Those equations represent linear-elastic material with sudden rupture where $\delta$ is displacement, $EA$ is bar stiffness, $L$ is bar length, $d_t$ is damage threshold in strain and $F_t$ is damage threshold in stress. Influence of the above equations on the load – displacement curve can be seen from Figure 4.

![Figure 4](image)

**Figure 4:** a) load - displacement curve for stiffness bar model, b) load - displacement curve for length bar model

In Figure 4 we could compare results for models with stiffness random generation vs. length random generation. Parameters for length stochastic model were $L_{\mu} = 1.0$, $\sigma_L = 0.25$, $d_t = 0.25$ and for stiffness stochastic model $EA_{\mu} = 1000.0$, $\sigma_{EA} = 500$, $F_t = 200$.

Stiffness stochastic model has been chosen for numerical experiments since it is considered more realistic; it is the directional stiffness that varies according to fiber orientation.
3.2 Model Distributions

Fiber stiffness is generated using Monte Carlo method and normal distribution as shown in Equation (2). An experiment with 500 generated bars produced mean bar stiffness $EA_{\text{mean}} = 942.7$ with standard deviation for bar stiffness $EA_\sigma = 495.5$ and load – displacement curve is in Figure 4.a.

In Figure 5.a there is contribution of some bars to the load – displacement curve and in Fig.5.b there is sorted contribution of all the bars at different displacement levels; at the displacement level $\delta = 0.1$ almost all the bars contribute to the load – displacement curve, at the displacement level $\delta = 0.3$ somewhat above 100 bars contribute to the load – displacement curve, etc.

Fiber angles are generated using Monte Carlo method with uniform pdf; for orientation, an experiment produced mean angle $\theta_{\text{mean}} = 3.103$ and analytic mean is $\theta_{\text{mean}} = \pi = 3.146$. Bar stiffness is generated from uniformly distributed angles using pdf from Equation (1); for orientation, an experiment with 1500 generated bars produced mean bar stiffness $EA_{\text{mean}} = 794.3$ and maximum possible stiffness was given $EA_{\text{max}} = 1250$.

In Figure 6.a there is load – displacement curve for the cosine distribution model and in Fig.6.b there is a normalized histogram for generated stiffness and its comparison with cosine distribution. There are not many parameters in this distribution that could serve fine – tune the load displacement curve.
There are some propositions how to introduce an additional variability, like considering the influence of granulometric diagram of concrete on angle distribution (there could be no fibers in the place where the aggregate is), but they will not be addressed here.

4 Inverse Stochastic Model

The main purpose of the inverse model is to enable reliable calculation of the model parameters from experimental results. That is enabled with introduction of order statistics that alleviates the need for Monte Carlo simulations. The parameter estimation is formulated as an optimization problem that is solved by applying the Levenberg – Marquardt procedure. The by-result of the procedure is sensitivity parameter so sensitivity analysis is also performed.

4.1 Levenberg – Marquardt Procedure

Optimization problem reads

\[ S_{err} = \sum_{im=1}^{nm} [F\delta_{im} - Fu(\delta_{im}, \sigma, \mu, d_t)]^2 \]  

where \( S_{err} \) is cumulative error, \( F\delta \) are measured values at measuring points \( im \) (from the experimentally determined load – displacement curve), and \( Fu(\delta_{im}, \sigma, \mu, d_t) \) are expected values from the stochastic model that are function of the parameters we would like to determine (mean, variance, damage threshold).

Minimization procedure leads to iterative explicit equations for each parameter calculation, e.g., for variance

\[ \Delta \sigma = \frac{\sum_{im}[F\delta_{im} - Fu(\delta_{im}, \sigma, \mu, d_t)]X\sigma_{im}}{\sum_{im}(X\sigma_{im})^2} \]  

where \( X\sigma \) is sensitivity parameter calculated at each measuring point. Procedure is iterative and parameter update is additive, \( \sigma_{i+1} = \sigma_i + \Delta \sigma \). Other parameters are calculated in a similar manner.

In Figure 7 there is comparison of experimental and model load - displacement curves for various values of model parameters.
Figure 7: comparison of experimental and model load - displacement curve for different a) stiffness variance, b) force damage threshold, c) stiffness mean
5 Sensitivity analysis

Sensitivity analysis is performed through numerical experiments performed for a range of parameter values. Figures present parameter sensitivity of normal distribution model; in order to emphasize the extremes, the error function is in semi-logarithmic scale.

In Figure 9 there is graphical representation of sensitivity analysis.

![Figure 8: sensitivity analysis for a) force damage threshold, b) stiffness mean](image)

From Figure 9 it is visible that the inverse procedure based on the Levenberg – Marquardt model is well posed and stable and all minima clearly visible. Evident is existence of some local minima for force damage threshold and stiffness mean value. Consequently, one may conclude that sensitivity analysis is necessary in parameter identification based on the above inverse procedure.

Examples presented throughout the paper are programmed and calculated in Mathcad\textsuperscript{8}.
6 Conclusion

This work is a proof of concept showing that parameters of fiber reinforced concrete could be obtained using inverse procedure that is formulated as a global optimization problem. The forward model is formulated using stochastic approach and order statistics. From the forward model, applying Levenberg - Marquardt method, produces an inverse model. The stable convergence has been obtained for all parameters: variability of fiber stiffness variance, variation in fiber mean stiffness and variation in damage threshold. However, there are some local minima for certain parameters and it is advisable to perform the sensitivity analysis to assure that global minima has been reached and that optimal value of parameters have been determined.

7 Acknowledgement

This research has been fully supported by Croatian Science Foundation under the project 9068. X-ray scans have been made at the shipyard “3-maj” Rijeka with help from mr. eng. Florian Sedmak. All help is fully appreciated.

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INFLUENCE OF STEEL FIBER CONTENT ON THE LOAD-BEARING CAPACITY OF ANCHORAGES IN CONCRETE

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ABSTRACT

In engineering practice, the number of applications is increasing, where relatively less brittle materials than normal concrete e.g. steel fiber reinforced concrete serve as base material for fastening systems. The beneficial effects of steel fibers in structural concrete such as limiting the crack widths, improving freeze-thaw resistance, improving impact and abrasion resistances, improving durability and increasing structural resistance are well known. However, the behavior of fasteners in steel fiber reinforced concrete is currently not regulated in the European Technical Approvals, and only a limited number of publications is available. There is a need to understand the behavior of fastening systems in such special concretes.

The aim of this study was to investigate the concrete cone capacity of individual anchors as well as anchor groups installed in steel fiber reinforced concrete. Numerous pull-out tests were carried out on single anchors and anchor groups in normal concrete and steel fiber reinforced concrete with a focus on the effect of steel fiber content on the concrete cone capacity. In the case of the group tests, the influence of load eccentricity and the influence of base plate thickness on the load redistribution were additionally investigated. Furthermore, a qualitative evaluation of the concrete fracture surface was conducted for a better understanding of the potential scatter of the test results caused by the unfavorable fiber orientation and distribution in the cement matrix.

The executed pull-out tests on single anchors and anchor groups indicated that the increased fiber content in the concrete has a positive effect on the load-bearing capacity of the fasteners, in general; the peak loads increased significantly due to the higher fiber content. The results of the group tests not only confirmed the increment of the peak loads, but also the load-displacement behavior of anchor groups in steel fiber reinforced concrete were influenced positively by the fibers. A better utilization of fastening systems could be attained due to the relatively ductile behavior of the steel fiber reinforced concrete.

1 Introduction

Although there is extensive research in the field of steel fiber reinforced concrete (SFRC) for structural and non-structural applications, only a few studies have focused on the performance of fasteners anchored in SFRC1,2. To date, the behavior of fasteners in SFRC is not regulated in the
European Technical Approvals. However, the number of applications is increasing where SFRC serves as base material for fastening systems. Therefore, the understanding of the modified performance of anchorages in SFRC is of particular interest. Examples for that include industrial floors, retrofitting of structures in seismic regions and machine fundaments where more ductile base material is needed than normal concrete due to the dynamic loads caused by machinery. In addition to structural safety reasons, economic reasons also call for using ductile and relatively ductile materials, since the thickness and the weight of the structural elements can be reduced by better material properties.

The aim of this study was to improve the current knowledge on the performance of single anchors and group of fasteners anchored in SFRC with respect to concrete cone failure. Within the scope of this research, the effect of steel fiber content on the concrete cone capacity was investigated in general. Furthermore, in the case of anchor groups subjected to tension loads the influence of load eccentricity and the influence of base plate thickness on the load-displacement behavior were investigated. The test program included centric and eccentric tension tests on individual anchors as well as on anchor groups in normal concrete and in SFRC.

2 Motivation

The steel fibers added to the concrete improve the fracture characteristics of the material. Due to addition of steel fibers, the overbridging of cracks (Figure 1) is ensured and so the cracking resistance of concrete increases. This beneficial effect leads to an increase in the concrete resistance under quasi-static, cyclic and impact loading and furthermore, to the increase of the spalling resistance. The idea in the background of using SFRC as base material for anchorages is the better utilization of fastening systems due to the relatively ductile fracture behavior and the crack-bridging ability of the SFRC in the case of concrete cone failure. In addition, the redistribution of load among the anchors within the group is ensured due to material properties of SFRC.

Figure 1: Crack-bridging mechanism a) in normal concrete and b) in SFRC

The improved properties of SFRC can be utilized in fastening technology if the targeted failure mode of anchorages subjected to tension loads is concrete cone. Fasteners transfer external loads into the base material via different mechanisms e.g. mechanical interlock, chemical bond, friction. The fastener, when loaded in tension, induces high tensile stresses in the concrete. After the developing tensile stresses exceed the tensile capacity of concrete in the load transfer zone, the crack initiates. This is followed by the crack propagation up to the peak load. The ultimate concrete cone capacity in normal concrete is generally obtained, when the resulting crack length reaches ca. 45% of the side
length of the final cone envelope. The concrete cone capacity is dependent on the concrete fracture energy, Young’s Modulus and embedment depth. If fasteners are anchored in SFRC instead of normal concrete, the propagation of the cracks can be slowed down according to Figure 2. If sufficient amount of fibers intercept the conical fracture surface, the concrete cone capacity may increase. However, the assumed better performance can be reached if the fiber distribution in the concrete is also optimal. After the crack initiation in SFRC, the overbridging fibers can still transmit tensile stresses over the crack into the surrounding concrete, so a stress-redistribution occurs. It is assumed that the total achievable increment of the concrete cone capacity due to the fiber content is limited by fiber rapture or fiber pull-out.

Figure 2: Crack propagation in the embedment zone a) in normal concrete and b) in SFRC

It is estimated that in the case of anchor groups subjected to tension loads, the fiber addition to the concrete has a greater positive influence on the anchor behavior than in the case of individual anchors. After the first cracks in SFRC, the stress redistribution can occur between the concrete and the fibers and thus among the adjacent individual anchors of the group as well.

According to EN 1992-4, sufficiently rigid anchor base plate must be used during the design of anchor groups in normal concrete. The base plate must remain elastic under design actions (Bernoulli hypothesis) and its deformation should be negligible compared to the axial anchor displacements of the individual anchors of the group (Figure 3a). If the applied base plate is not sufficiently rigid, then the deformation of the plate has an influence on the lever-arm of the internal forces, which must be taken into account during the calculation (Figure 3b).

Figure 3: Centric tension loading on a 1x3 anchor group using a) rigid base plate and b) non-rigid base plate

Figure 4: Eccentric tension loading on a 1x3 anchor group using a) rigid base plate and b) non-rigid base plate
The individual anchors of an anchor group are subjected to different axial tension forces if (1) centric tension load is applied on the anchor group, but non-rigid base plate is used (Figure 3b) or (2) the applied load is eccentric on the anchor group in the case of both rigid and non-rigid base plates (Figure 4). Anchors, which are closer to the point of the load application have to take up higher forces than those located farther. An anchor group fails by a common concrete cone breakout, provided that the full tensile capacity of the concrete is utilized. On the contrary, in the case of an anchor group installed in SFRC, the redistribution of forces due to transmitting tensile stresses over the cracks by the fibers and the redistribution among the adjacent individual anchors is possible within the group. Due to the load redistribution among the adjacent anchors, and due to the delayed activation of the anchors, increased load-carrying capacity and increased ductility of the anchor group can be achieved.

Based on these considerations, the eccentricity of the tension loading and the base plate stiffness should have a lesser negative influence of the load bearing-capacity of anchor groups in SFRC than in normal concrete.

3 Experimental Investigations

3.1 Overview

In this study, pull-out tests in normal concrete and in SFRC were carried out on single anchors and anchor groups with a focus on the influence of fiber content, eccentric loading and anchor plate stiffness in the case of concrete cone failure. An injection system with relatively high bond strength was chosen to transfer the loads into the concrete by means of bond. The test parameters like concrete strength, diameter of the threaded rod, embedment depth and anchor spacing (group test) were selected such that all tested anchors and anchor groups failed by concrete cone. The test program was made in a way that the discussed assumptions regarding the influence of group effect, load eccentricity and base plate stiffness on the ultimate concrete cone capacity could be investigated (Table 2).

3.2 Base Material

In general, the design of a SFRC mixture can be made in accordance with the design provisions for conventional concrete, provided that the fiber content does not exceed 40-50 kg/m³ in the mixture and the fibers can be considered as additional aggregates in the cement matrix. In the case of SFRC, the same equipment which is used for conventional concrete production can be used for mixing and finishing to ensure the proper compaction. In this study, the production of the normal concrete and SFRC test members was carried out in two steps. Firstly, the normal concrete (Basic mixture Table 1) was mixed and the test members were cast. Afterwards, the required amount of steel fiber was calculated by taking into account the remaining basic mixture in the concrete mixer. The fibers were dispersed uniformly to the freshly mixed basic concrete mixture. To ensure the clump-free state of fibers in the mixture, the mixer was rotated at full speed continuously for approximately 3 minutes. The SFRC specimens were cast, subsequently. By this process, the same basic mixture could be used for normal concrete and SFRC thus enabled the direct comparison of test results in normal concrete to those of SFRC.
To ensure sufficient bonding between the concrete matrix and fibers i.a. the maximum grain size \( d_{\text{max}} = 8 \text{ mm} \) should not exceed one-third of the fiber length. Hooked-end fibers HE 75/3510 of the company ArcelorMittal were used. This fiber has a relatively high tensile strength (1200 N/mm\(^2\)) and its bond, which is dependent on the aspect ratio to the matrix (L/d) is enhanced additionally by the mechanical interlock due to the end-hooks. The nominal wire diameter of the fiber was \( d = 0.75 \text{ mm} \) and the nominal fiber length was \( L = 35 \text{ mm} \) (Aspect ratio \( L/d = 47 \)).

### Table 1: Concrete mixture

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>Cement CEMII/A-LI 42,5R [kg/m(^3)]</th>
<th>Aggregate 0-2 mm [kg/m(^3)]</th>
<th>Aggregate 2-8 mm [kg/m(^3)]</th>
<th>Water [kg/m(^3)]</th>
<th>w/c [-]</th>
<th>Fiber content [kg/m(^3)]</th>
<th>Super-plasticizer [M% of Cement]</th>
<th>Retarder [M% of Cement]</th>
<th>Compressive strength ( f_{c,c} [\text{N/mm}^2] )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basic mixture</td>
<td>337,7</td>
<td>831,4</td>
<td>997,1</td>
<td>182,2</td>
<td>0,54</td>
<td>0</td>
<td>0,86</td>
<td>0,3</td>
<td>67</td>
</tr>
<tr>
<td>SFRC</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>50</td>
<td>0,86</td>
<td>0,3</td>
<td>65</td>
</tr>
</tbody>
</table>

#### 3.3 Anchor System

The tension tests on individual anchors and anchor groups in normal concrete and in SFRC were carried out with an injection system (Epoxy mortar + steel element) with relatively high bond strength to avoid the pull-out failure. The bond strength of the injection system was approx. 32 N/mm\(^2\) (determined on the basis of confined tension test results in non-cracked normal concrete). The steel element was threaded rod in size M16 and strength class 12.9. The installation of the injection system was performed according to the corresponding Manufacturer’s Installation Instructions. Since the imprecisely positioned anchors would falsify the results in the case of group tests, an accurate positioning of the anchors during the installation was necessary. To this end, a steel template with pilot-holes was built with accurate hole spacing for the drilling process and for keeping the anchors in position during the hardening of the mortar.

#### 3.4 Test Program

Tension tests were carried out on individual anchors and anchor groups with 3 anchors in a row with varied base material (normal concrete and SFRC) and varied eccentricity of loading (\( e = 0 \text{ mm} \); \( e = 60 \text{ mm} \); \( e = 120 \text{ mm} \)), varied base plate thickness (\( t = 50 \text{ mm} \); \( t = 25 \text{ mm} \)), respectively. The edge distances \( c > c_r \) (no edge influence) and the anchor spacing \( s = 120 \text{ mm} \) (\( s < s_{c,N} \); group effect) were kept constant in this study. The geometry of the base plate and anchor group were designed such that the assumptions of the study could be verified. The test program and the test parameters are summarized in Table 2.
Table 2: Test program

<table>
<thead>
<tr>
<th>Test type</th>
<th>Configuration</th>
<th>Embedment depth $h_e$ [mm]</th>
<th>Anchor spacing $s$ [mm]</th>
<th>Edge distance $c$ [mm]</th>
<th>Eccentricity of loading $e^*$ [mm]</th>
<th>Base plate thickness $t$ [mm]</th>
<th>No. of tests in normal concrete</th>
<th>No. of tests in SFRC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Individual</td>
<td>1</td>
<td>70</td>
<td>-</td>
<td>$&gt;c_{cr}$</td>
<td>0</td>
<td>50</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Group</td>
<td>1x3</td>
<td>70</td>
<td>120</td>
<td>$&gt;c_{cr}$</td>
<td>0</td>
<td>50</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>Group</td>
<td>1x3</td>
<td>70</td>
<td>120</td>
<td>$&gt;c_{cr}$</td>
<td>120</td>
<td>25</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>

*Eccentricity of loading "e" is defined in this study as the distance between the point of the load application and the center of gravity of all anchors within the group.

3.5 Test Setup and Testing

The tension tests on individual anchors and anchor groups were carried out according to the recommendations of ETAG 001, Annex A. The test setup consisted of a tension test rig with adequate spacing to allow the formation of an unrestricted concrete cone, a hydraulic cylinder, a calibrated load cell, a universal joint, fixture/anchor plate and calibrated displacement transducers. The load was actuated using a hydraulic test cylinder with respect to the expected load range. The applied loads, the vertical anchor displacements and the anchor plate displacements (at the point of load application) were measured and recorded via Spider8 using the commercial data acquisition software DiAdem. Furthermore, the axial anchor loads were measured on the individual anchors within the anchor groups using HBM force washers. For the group tests, anchor plates with pre-drilled internally threaded holes were used to transfer the load to the anchor plate by the loading threaded rod. In the case of group tests with eccentric loading, a special hinge was connected to the anchor plate via threaded rod to allow nearly free rotation of the anchor plate along its longitudinal axis (Figure 5 and Figure 6). The distance between the hinge and anchor plate was 5 cm during the testing. The lowering of the hinge position was in this study not possible because of the anchor configuration (small spacing) and testing conditions.

![Figure 5: Load application in the case of eccentric loading](image-url)
4 Results

4.1 Overview

In this study, pull-out tests in normal concrete and SFRC were performed on individual anchors and anchor groups according to Table 2. With the systematic variation of the parameters, it was possible to investigate the influence of eccentric loading and the influence of anchor plate stiffness on the concrete cone capacity. Besides the load-displacement curves of each executed test (Figure 7, Figure 8), the relative increment of ultimate concrete cone capacities in function of the investigated test parameters are also presented for better graphic interpretation purposes. Additionally, qualitative and quantitative evaluation of the concrete fracture surface was conducted for the better understanding of the scatter of the test results caused by the unfavorable fiber orientation and distribution in the cement matrix. The results of the pull-out tests are summarized in Table 3.

Table 3: Test results

<table>
<thead>
<tr>
<th>Test type</th>
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Figure 7: Load-displacement curves of the centric tension tests on individual anchors

Figure 8: Load-displacement curves of the tension tests on anchor groups a) centric, rigid plate, b) eccentric (1), rigid plate; c) eccentric (2), rigid plate; d) eccentric (2), non-rigid plate

4.2 Influence of Fiber Content

The tension test results on single anchors and anchor groups confirmed the expected beneficial effect of the fiber content on the concrete cone capacity of the fasteners. The peak loads and the ductility increased due to fiber addition. In the case of individual anchors, an increase of 18% in the average peak loads was observed due to 50 kg/m³ fiber addition, whereas the results of group tests showed an increase in the average failure loads in the range of 25-42%, depending on the eccentricity of loading and anchor plate thickness (see Figure 9). That means that the fiber addition to the concrete has a
more significant effect on the load-displacement behavior of fasteners if anchor groups instead of single anchors are fastened in SFRC. The fibers influence the tensile behavior of the concrete. However, not the tensile strength, but the cracking resistance of the concrete i.e. ductility, fracture toughness is affected by the fibers in particular. This influence is strongly dependent on the fiber content, fiber orientation and the fiber type. In the case of anchor groups subjected to tension loads in SFRC is the redistribution of forces due to transmitting tensile stresses over the cracks by the fibers and due to the redistribution among the adjacent individual anchors is within the group possible. These two mechanisms are superposed and thus lead to a better utilization of anchors if they are applied as anchor groups instead of individual anchors. However, the increase of the peak loads depends on the eccentricity of loading and the anchor plate stiffness.

![Figure 9: Influence of fiber content on the ultimate loads in the case of a) individual anchors and b) anchor groups](image)

![Figure 10: Typical concrete cones of individual anchor and groups in SFRC and in normal concrete](image)

### 4.3 Influence of the Load Eccentricity

The test results on anchor groups subjected to centric and eccentric tension loads show that the negative influence of eccentricity on the load-displacement behavior of anchor groups is somewhat attenuated in case of the anchor groups in SFRC compared to normal concrete. It can be seen in Figure 11 that the ratio $N_{u,\text{SFRC}} / N_{u,\text{Normal}}$ increases with increasing eccentricity. This can be explained by the redistribution of loads due to the fibers’ crack overbridging properties and by the redistribution of the forces among the adjacent anchors within the group. The load redistribution among the anchors could be optimized by the fibers. This led to the fact that at a definite point in time during the loading a larger concrete area was activated and therefore, higher peak load could be reached.
4.4 Influence of Anchor Plate Stiffness

The influence of the anchor plate thickness on the load-displacement behavior was tested on eccentrically loaded anchor groups (the group was loaded at the position of the outer anchor) in normal concrete and in SFRC. For the rigid anchor plate $t_1=50$ mm was chosen. To investigate the influence of the anchor plate stiffness, the plate thickness was reduced to $t_2=25$ mm (non-rigid plate). The ratio $t/s$ of the rigid plate was 0.4 and 0.2 for the non-rigid plate, respectively. It is demonstrated in Figure 12 that due to the reduction of the anchor plate thickness, the peak loads decreased by 12% in normal concrete and only 1% in SFRC. This shows that the anchor plate thickness has less influence on the load-displacement behavior of anchor groups in SFRC than in normal concrete. The individual anchors of the tested anchor groups using non-rigid anchor plate and eccentric loading received different tension loads, and the anchors closer to the load application were loaded higher than those, which are farther. After the (most loaded) outer anchor of the group has failed in normal concrete, the peak load could not be increased further, because the capacity of the other neighboring anchors only influenced the post-peak behavior of the group and they did not contribute to the ultimate load. On the contrary, in the case of anchor groups in SFRC, an increment of the ultimate group capacity was observed after the first cracking. This is explained by the load redistribution among the adjacent individual anchors within the group due to the fibers’ crack overbridging mechanism. The material properties of SFRC and ductility of the anchor plate enabled the redistribution of the loads without falling of the L-D curve and the activation of the adjacent anchors could contribute to the ultimate group capacity.

Figure 12: Influence of anchor plate stiffness on the ultimate loads in the case of anchor groups
5 Conclusion

In this study the concrete cone capacity of anchorages was investigated in non-cracked normal concrete and in SFRC. The test program was created in a way that the influences of fiber content, group effect, eccentric loading, anchor plate stiffness on the performance of anchorages could be studied. The test program included reference pull-out tests on individual anchors and centric and eccentric tension tests on anchor groups with rigid and non-rigid anchor plates in normal concrete and in SFRC. The positive effect of steel fiber content on the concrete cone capacity was verified on single anchors and on anchor groups as well. A better utilization of fastening systems could be attained in SFRC due to its relatively ductile (compared to normal concrete) fracture behavior. However, the positive influence of the fibers is more pronounced in case of anchor groups compared to individual anchors. Due to the group effect and the crack overbridging mechanism of the fibers, anchor groups in SFRC showed a higher relative increase in their ultimate loads than the individual anchors. The group test results verified that not only the peak loads, but also the load-displacement behavior of anchor groups in SFRC are influenced positively by the fibers. Furthermore, the research confirmed that the negative influence of the load eccentricity and the negative influence of anchor plate thickness on the load-displacement behavior of anchor groups is less pronounced in case of anchor groups installed in SFRC instead of normal concrete.

Although, the results of tests performed in steel fiber reinforced concrete are promising, more research is needed on different anchor configurations with varying the anchor spacing and eccentricity to generalize the findings of this study. The anchor plate thickness should be investigated by parameter studies with different t/s ratios. In addition, it has to be taken into account that this study focused on the concrete cone failure of fasteners in SFRC with hooked-end fibers and for this purpose the test parameters and anchor type were selected so that all other failure modes were unlikely to occur. For this reason, future studies need to be performed with particular interest on the application of different anchor types in SFRC with diverse fiber types.

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17. DIN EN 12390-1:2012-12 Prüfung von Festbeton – Teil 1: Form, Maße und andere Anforderungen für Probekörper und Formen; Deutsche Fassung EN 12390-1:2012


RESIDUAL TENSILE CAPACITY OF POST-INSTALLLED ANCHORS
AFTER EXPOSURE TO FIRE

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ABSTRACT

The fire performance of various types of fastenings is generally described in terms of fire resistance, which defines the time until failure due to combined effects of fire and mechanical load. However, there are only very few results available related to the residual tensile capacity of the anchors after exposure to fire. Furthermore, existing results are primarily obtained for headed studs, without considering the interaction of heating with the different mechanisms of load transfer present for various types of the post-installed mechanical anchors.

In the present paper, the results of an experimental campaign on the residual pull-out (tensile) capacity of two anchor types, namely concrete screws and undercut anchors, after exposure to fire are presented and discussed. The anchors were installed in normal strength concrete, whereby for both the anchor types three different effective embedment depths ($h_{ef} = 40$ mm, $60$ mm and $80$ mm) were investigated. The concrete specimens were then exposed to standard fire (according to ISO 834) for a duration of 60 minutes. After cooling down to room temperature, the residual tensile capacity of the two investigated anchor types was measured. Post-fire performance of the anchors was compared to that of the reference specimens, which were not exposed to fire. It was found that the two anchor types exhibit significant differences in the post-fire performance. These results underlay the need to differentiate between various types of fastenings when considering the behavior during and after fire.

1 Introduction

Anchorages using post-installed anchors form a versatile means of connection between a component and the concrete structure. The post-installed anchors may transfer the applied tensile loads through the mechanism of mechanical interlock (e.g. undercut anchors and concrete screws), friction (e.g. expansion anchors), bond (e.g. bonded anchors) or a combination thereof. The mechanical behavior of these anchors has been significantly researched over the past decades\textsuperscript{1} that has led to the development of standards\textsuperscript{2,3} and guidelines\textsuperscript{4} for safe design of anchorages using these anchors. However, the research work to investigate the influence of fire on the performance of these anchorages is rather limited.
It is common for a structural fire to begin with burning and falling of the claddings and attachments fastened to the main concrete structure. Although the fire resistance of the attachments itself is of great importance, the fire resistance of the mechanical fasteners is vital for fire safety of the whole structure. Unfortunately, there are no unified fire test procedures and no commonly accepted fire design rules for fastening systems till now. The current EN1992-4, Annex D gives only informative recommendations for reduction of anchor resistance for the fire exposure up to 90 minutes and 120 minutes, and in ACI 3183 code anchor resistance under fire is not yet considered.

In this work, the residual pull-out (tensile) capacity of post-installed mechanical anchors after exposure to fire is experimentally investigated. Two anchor types, namely concrete screws and undercut anchors, both transferring tension loads through mechanical interlock, were used for these investigations. The anchors were installed in normal strength concrete, with three different effective embedment depths (h_{ef} = 40 mm, 60 mm and 80 mm) for both the anchor types. The concrete specimens were exposed to standard fire (according to ISO 834) for a duration of 60 minutes. After cooling down to room temperature, the residual tensile capacity of the two investigated anchor types was measured. Post-fire performance of the anchors was compared to that of the reference specimens, which were not exposed to fire. Three tests per case were performed. This paper presents and discusses the results of the research work.

2 Literature review

The strength of concrete is significantly influenced by the exposure to high temperature and fire. This leads to strong influence on the capacity of connections between steel and concrete, e.g. the load-bearing behavior of fasteners in concrete corresponding to concrete breakout failure. In case of fire exposure, due to high thermal gradients generated in a short span of time, the concrete gets damaged, which can result in strong reduction of failure capacity of fasteners. Intuitively, the anchors installed close to an edge or at shallow embedment depth would suffer stronger degradation due to fire compared to those installed away from the edges and at deeper embedment depth.

Reick performed experimental and numerical analyses on the fire resistance of undercut and headed stud anchors under fire associated with the concrete cone failure under tension loads. It is reported that the diameter of the concrete cone developed in case of tests after fire is larger than the diameter of the fracture cone obtained for the corresponding case at ambient temperature. The relatively rapid temperature decrease with increasing depth in the concrete corresponded to the increase of the relative concrete capacity with increasing embedment depth of the anchor. Ožbolt et al. carried out numerical simulations based on 3D thermo-mechanical FE modelling on headed stud anchors with different embedment depths exposed to fire and loaded in tension. It was demonstrated that the concrete cone resistance of anchors with relatively small embedment depth can be significantly reduced due to fire exposure. For anchors with large embedment depth, compressive stresses may be generated, which can lead to no reduction of resistance or even slightly higher capacity due to restraining conditions of concrete under fire.

Bamonte et al. conducted experimental research on the tensile capacity of undercut anchors with different embedment depths installed in thermally damaged concrete. Results showed that after
reaching only 400 °C the pull-out capacity was roughly 20% of the values obtained from reference tests without high temperature exposure.

Periškić\textsuperscript{11} performed experimental tests and numerical simulations on the load-bearing behaviour of single anchors and anchor groups located close to concrete edge and away from concrete edge under tensile load at high temperatures, considering the fire exposure on one side and two sides of the edge. It was shown that for anchors located away from concrete edge, the embedment depth of anchor plays a main role on the residual capacity of anchors loaded in tension under fire exposure, the larger the embedment depth is, the smaller the reduction in the failure load. When the anchor is located close to an edge, the two-sided fire on the edge induces strong reduction of the tensile capacity under fire exposure, which is reasonable due to the thermal penetration of heat into concrete member from both sides of the edge.

In this work, experimental investigations are carried out on the influence of fire on the tension behavior of two different types of post-installed anchors namely undercut anchors and concrete screws installed at different embedment depths.

3 Experimental investigations

3.1 Anchors

The post-installed anchors from company fischer, fischer-Zykon-Anchor (FZA) and fischer concrete screws (FBS) were used for the tests in this work. The anchors were installed at three different effective embedment depths of 40 mm, 60 mm and 80 mm.

3.2 Test specimen

Square shaped concrete slabs (1.2m x 1.2m x 400mm) were used as the test specimens with the anchors installed in a way that a space of at least 4 times the effective embedment depth was available freely around each anchor without any interference from the space required for the other anchor. The test specimens before and during the fire test are shown in Figure 1.

![Figure 1: Test specimens with installed anchors before and during the fire test](image-url)
Normal strength concrete of grade C20/25 was used in the tests.

### 3.3 Test program

The test program consisted of testing two different anchor types installed at three different embedment depths tested under tension loads in ambient conditions and residual conditions after an exposure to 1 hour of ISO 834 standard fire. For each case three specimens were tested to verify repeatability. The test program is summarized in Table 1.

Table 1: Test program for bond behavior of reinforcement in concrete under elevated temperature

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<td>After 1hr fire</td>
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### 3.4 Test procedure and setup

#### 3.4.1 Fire treatment

For heating, the test specimens were placed inside a fire oven with oil burners in a way that only one face of the concrete slab was exposed to fire. The other faces were insulated using mineral wool and aerated concrete blocks. During the fire test, the oven temperature was measured and controlled using plate-thermocouples. Upon finishing the heating phase, the ventilation ducts were opened and the specimens were allowed to cool down without opening the oven.

#### 3.4.2 Pullout tests

The tension pullout tests were performed using the standard unconfined test setup with a clear support distance of 4 times the effective embedment depth of the anchor. The typical test setup for the pullout tests is shown in Figure 2.

Figure 2: Test setup employed for performing pullout tests
4 Test results

4.1 Undercut anchors

4.1.1 Embedment depth = 80 mm
The load-displacement curves obtained from the pullout tests performed on the undercut anchors with embedment depth, $h_{ef} = 80$ mm in ambient and residual conditions are plotted in Figure 3. The reference tests displayed a ductile behavior after reaching the peak load typical of steel failure, which was the obtained failure mode for the tests in ambient conditions. The mean peak resistance from the reference tests was obtained as 66.0 kN. After fire exposure, the failure mode changed to concrete cone breakout. The load-displacement curves displayed relatively low stiffness and quasi-brittle load-displacement curves. Mean peak residual capacity after fire exposure was obtained as 15.6 kN.

4.1.2 Embedment depth = 60 mm
The load-displacement curves obtained from the pullout tests performed on the undercut anchors with embedment depth, $h_{ef} = 60$ mm in ambient and residual conditions are plotted in Figure 4. In this case, both the reference tests as well as the tests in residual conditions displayed a gradual drop in the load after reaching the peak load typical of concrete cone breakout, which was also the obtained failure mode for the tests in ambient conditions and in residual conditions. The mean peak load from the reference tests was 35.2 kN and from the residual tests was 12.6 kN.

4.1.3 Embedment depth = 40 mm
The load-displacement curves obtained from the pullout tests performed on the undercut anchors with embedment depth, $h_{ef} = 40$ mm in ambient and residual conditions are plotted in Figure 5. As expected, the load-displacement curves are most severely affected by fire in this case, with the mean peak load for the reference tests being 26.1 kN and that for residual tests being 3.7 kN. Concrete cone failure occurred in all the tests in reference as well as residual conditions.

The failure modes obtained from the pullout tests on undercut anchors are summarized in Figure 6.
Figure 4: Load-displacement curves obtained from pullout tests on undercut anchors, $h_{ef} = 60$ mm

Figure 5: Load-displacement curves obtained from pullout tests on undercut anchors, $h_{ef} = 40$ mm

Figure 6: Failure modes obtained from pullout tests on undercut anchors
4.2 Concrete screws

4.2.1 Embedment depth = 80 mm

The load-displacement curves obtained from the pullout tests performed on the concrete screws with embedment depth, $h_{ef} = 80$ mm in ambient and residual conditions are plotted in Figure 7. Unlike undercut anchors, concrete screws failed by combined concrete cone and pullout failure in the reference as well as residual tests. The mean peak failure load for the tests in ambient conditions was obtained as 49.0 kN and that for tests in residual conditions was obtained as 16.3 kN.

![Figure 7: Load-displacement curves obtained from pullout tests on concrete screws, $h_{ef} = 80$ mm](image)

4.2.2 Embedment depth = 60 mm

The load-displacement curves obtained from the pullout tests performed on the concrete screws with embedment depth, $h_{ef} = 60$ mm in ambient and residual conditions are plotted in Figure 8. All the tests in ambient and residual conditions failed by combined concrete cone and pullout. The mean peak failure loads for reference and residual tests were obtained as 30.9 kN and 6.7 kN respectively.

![Figure 8: Load-displacement curves obtained from pullout tests on concrete screws, $h_{ef} = 60$ mm](image)
4.2.3 **Embedment depth = 40 mm**

The load-displacement curves obtained from the pullout tests performed on the concrete screws with embedment depth, $h_{ef} = 40$ mm in ambient and residual conditions are plotted in Figure 9. All the tests failed in combined concrete cone and pullout failure. The mean peak failure loads for reference tests and residual tests were obtained as 15.1 kN and 3.2 kN respectively.

![Figure 9: Load-displacement curves obtained from pullout tests on concrete screws, $h_{ef} = 40$ mm](image)

The failure modes obtained from the pullout tests on undercut anchors are summarized in Figure 10.

![Figure 10: Failure modes obtained from pullout tests on concrete screws](image)

4.3 **Summary**

The mean reference and residual peak failure loads obtained for the undercut anchors and concrete screws are plotted as a function of the embedment depth of the anchors in Figure 11. In general, the peak loads obtained by the concrete screws were less than the peak loads obtained by the undercut anchors. This is attributed to the failure modes obtained as combined concrete cone and pullout for
the concrete screws and concrete cone failure (or steel failure for the case of $h_{ef} = 80$ mm) for the undercut anchors (Figure 12). However, after exposure to fire, the failure loads for both the anchor types were similar for corresponding anchor depth.

![Figure 12: Typical failure modes obtained from the reference pullout tests on concrete screws](image)

An interesting observation made from the inspection of the breakout surface was that after exposure to fire, several radial cracks were passing through each anchor hole. This is due to the fact that the anchor borehole acts as a notch (discontinuity) and attracts cracks.

![Figure 13: Radial cracks originating from the anchor borehole observed in the residual tests](image)
5 Conclusions

The residual pull-out tests on concrete screws and undercut anchors after exposure to fire showed a significant influence of the exposure to fire on residual tensile load carrying capacity of the anchors. It was observed that at larger embedment depths, undercut anchors retain a higher percentage of their reference failure load compared to concrete screws, while for smaller embedment depth, the concrete screws retain a higher percentage of their reference load-carrying capacity. Due to exposure to fire, the concrete gets damaged, its mechanical properties reduce and the anchor hole is intercepted by several cracks, which negatively affects the behavior of the anchors under tension forces. More tests on different anchor types and embedment depths are needed to clearly bring out the influence of fire on different types of anchors installed at different embedment depths.

References:

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6. Schneider U. Verhalten von Beton bei erhöhten Temperaturen (Behaviour of concrete at high temperatures) (in German). Dtsch Ausschuß Für Stahlbet 1982;337
INVITED LECTURES

SPECIAL SESSION: 75\textsuperscript{TH} BIRTH ANNIVERSARY
OF PROF. ROLF ELIGEHAUSEN
ROLE OF FIBER REINFORCEMENT ON LAP SPLICES BEHAVIOR

Giovanni Plizzari$^{1*}$ and Giovanni Metelli$^{1}$

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ABSTRACT

Bond behavior involves the formation of radial transversal pressure as a result of the wedge action of the crushed concrete between the bar ribs. The latter may cause the onset of longitudinally splitting cracks when the tensile strength is reached in the concrete cover. The propagation of splitting cracks, and thus the brittle failure of the anchorage or a splice, can be efficiently counteracted by the confinement action provided by stirrups and/or by the post-cracking residual strength of fiber reinforced concrete (FRC), which allows a more uniform distribution of bond stress along a splice. Several full-scale beams reinforced with rebars of different diameters were tested with different percentages of longitudinal reinforcement lapped at mid-span. A traditional volume fraction of hooked steel fibers (equal to 0.5%) was adopted for FRC. Experimental results show that the toughness of FRC can enhance the behavior of the weak joint with only lapped bars. The capability of FRC to control the flexural and splitting crack opening and propagation can increase the strength of staggered lapped splices, thus allowing a reduction of lap length.

1 Introduction

Bond behavior mainly depends on the mechanical interaction between concrete and bar ribs. This mechanism involves the formation of radial transversal pressure as a result of the wedge action of the crushed concrete between the ribs$^{1,2,3}$. The latter may cause the onset of longitudinally splitting cracks when the tensile strength is reached in the concrete cover$^{4,5,6}$. The propagation of splitting cracks, and thus the brittle failure of anchorages or splices, can be efficiently counteracted by the confining action provided by transverse reinforcements$^{5,7}$, by transversal pressure$^{8,9}$ or by cohesive stresses between the surfaces of splitting cracks$^5$. The latter contribution depends mainly on the post-cracking resistance of concrete$^{10}$, which can be significantly enhanced by the addition of fibers to the concrete mix (Figure 1).

The availability of building codes makes nowadays Fiber Reinforced Concrete (FRC) a structural material in many Countries. After several decades of research studies, design rules for FRC are now available for several structural elements, including beams, slabs or precast elements as tunnel segments. Unfortunately, design rules, so far, do not consider the benefits of FRC on bond, anchorages and splices.

As a matter of fact, several studies$^{11,12,13,14}$ have shown a significant increase of bond strength when fibers are included in the concrete matrix. In fact, the addition of fibers increases the confinement effect of concrete cover, which withstands the propagation of splitting cracks. The research of
Chao\textsuperscript{15}, based on pull-out tests in strain hardening FRC composites, proved the higher efficiency of twisted steel fibers (with a volume fraction of 2\%) in providing confinement action and crack control, as compared to an equivalent amount of transversal reinforcement. Other studies have demonstrated a lower efficiency, due to a workability problem of FRC near the bar ribs\textsuperscript{16}; a main aspect is also represented by a possible orientation of fibers (due to the concrete flow) with respect to the main splitting crack (Figure 1).

![Splitting failure in FRC](image1)

**Figure 1: Effect of fiber reinforcement on bond behavior.**

One of the main bond issues in practical applications is represented by the lap splices as a coupler system to link the steel rebars in concrete members. Lee\textsuperscript{17} investigated the behavior of lap-spliced reinforcements with a concrete cover of 2.5 or 3.3 times the bar diameter ($d_b$) in Ultra High Performance FRC (UHPFRC), with a compressive strength of 130 MPa, by varying the volume of steel fibers from 0 to 2\%. The results of four-point bending tests on seven beams showed that the lap-splice was able to develop the bar yield strength when the volume fraction ($V_f$) of fibers was greater than 1\% or 2\%, for a lap length ($l_b$) of 10$d_b$ or 5$d_b$, respectively. Lageir et al.\textsuperscript{18} carried out direct tensile tests on tensile lap splices of large reinforcing bars ($d_b=25$ or 33 mm) with a small concrete cover ($c=1.2d_b$) in UHPFRC ($f_{cm}=104\div167$ MPa) with volume fraction of straight steel fibers up to 4\%. The strain hardening behavior of UHPFRC led to a significant increase in the lap strength, of 53\%, when the fiber content increased from 1\% to 4\%, with a splice length of 10$d_b$. The yield stress of 400 MPa could be ensured in short laps ($l_b=12d_b$) with a fiber content of 4\%. In relation to High-Performance FRC (HPFRC), Hamad et al.\textsuperscript{19} investigated full-scale beams with ribbed bars lapped over a length of 305 mm, with the aim of evaluating the effect of bar diameter ($d_b=20$, 25 or 32 mm) and the content of hooked steel fiber on the lap strength. The results showed an increase in bond strength of about 50\% for $V_f=2\%$, in comparison with plain concrete (PC), and the remarkable capability of fibers to mitigate the size effect of bond. Tests on 32 mm and 20 mm lapped bars in FRC specimens with 0.5\% volume fraction of fibers provided a bond strength 33\% and 10\% higher, respectively, than the reference specimens of plain concrete.

Harajili et al.\textsuperscript{20} investigated the behavior of short lap splices ($l_b=5d_b$) with a small concrete cover ($c<2.1d_b$) and a $V_f$ ranging from 0.5 to 2\%, by varying the bar diameter (from 16 to 32 mm). Tests on beam specimens in normal strength concrete showed a 26\% and 33\% increase in the splitting bond strength for a $V_f$ of 1\% and 2\%, respectively. A 0.5\% volume fraction of fibers had a beneficial
effect on the splitting strength of large bars (32 mm) only. A significant improvement in post-peak behavior, in comparison to plain unconfined concrete, was also observed.

The main building Codes for concrete structures encourage staggering of lap splices since the lapped bars are considered weak joints, which may impair the safety and the ductility of the RC member. When staggered lap splices are used, splice strength may be increased by the enhanced confinement of the concrete cover because the bars are widely spaced. Moreover, the brittleness of the joint may be reduced because the continuous bars can still guarantee part of the flexural strength of the section if splice failure occurs.

For this reason, several structural codes require a longer lap length if all the bars are lapped at one section. According to ACI 318-11\textsuperscript{21}, when the percentage of bars spliced is greater than 50%, the splice length should be at least 1.3 times the anchorage length (splice of Class B). Eurocode 2\textsuperscript{22} is more conservative and requires lap lengths to be increased by a factor of up to 1.5 times the anchorage length, depending on the proportion of bars lapped at the section. The new fib Model Code 2010\textsuperscript{23} for concrete structures does not distinguish between anchorages and laps. A lap length depends on the concrete strength, concrete cover, bar diameter, transverse reinforcement and transverse pressure; moreover, the dependency of the lap length on the percentage of lapped bars was removed. On the basis of the experimental results on staggered lapped bars from Cairns\textsuperscript{24} and Metelli et al.\textsuperscript{25}, the joint strength did not benefit from staggering lap splices and a significant reduction of the joint strength (by about 25% when the percentage of lapped bars reduces to 50%) was measured. This “weakening effect” of staggered laps was explained by the higher stiffness of a pair of lapped bars, in comparison with a continuous bar, which causes the lap splice to attract a greater share of the total tension force in the reinforcements (the usual assumption is equally distributed tensile action among continuous and lapped bars within the joint).

While many studies are available in the literature on the effect of types and dosage of fibers on bond strength, few experimental results have been published to date on the behavior of lap splices in traditional FRC with a volume fraction ($V_f$) lower than 1%, which is typical of many structures. For this reason, an experimental campaign was carried out at the University of Brescia on lap-splices in FRC beams with a low volume fraction of fibers, significant for many practical applications\textsuperscript{26}. Beam specimens were designed by varying the percentage of lapped bars at a section, with the aim at understanding the potential capacity of the post-cracking residual strength of FRC to enhance splice behavior.

2 Experimental program

2.1 Specimens details and materials

Two test series were carried out on ten beams, designed with lap-splice at mid-span, to be tested under constant bending moment over the joint length. The beams were 4.5 m long, 0.35 m deep and 0.30 m wide. The first series consisted of four beams with three longitudinal bars with a diameter ($d_b$) of 20 mm while, in the second series, six beams were reinforced with four bars with a diameter of 16 mm. The reinforcement ratio ($\rho_s$) was equal to 1% and 0.9% in the beams of the first and of the second series, respectively. As depicted in Figure 2, each series included a reference beam with continuous reinforcements and a beam with all bars lapped at mid-span in plain concrete, while in
the FRC beams, the laps were tested varying the percentage of bars lapped at a section. In the first series, the FRC specimens were tested with either one or all three 20 mm bars lapped, corresponding to 33% or 100% of bars lapped at a section, respectively. When using 16 mm rebars (series 2), FRC specimens were made with one, two or all four bars lapped, corresponding to a percentage of laps (with respect to the total number of rebars) of 25%, 50% or 100%. Beams with two 16 mm bars lapped were designed with splices placed both internally and externally (in the section).

By assuming a target compressive concrete strength ($f_{cm}$) of 28 MPa (representative of a C20/25 concrete class) and the steel yield strength ($f_{ym}$) of 550 MPa, according to the new fib-MC2010 (Eq. 1) a splice length ($l_b$) of 25 bar diameters ($d_b$) provides a maximum bar stress ($f_{stm}$) similar to the yield stress with a splitting-failure mode (Table 1).

$$f_{stm} = 5.4 \left( \frac{f_{cm}}{25} \right)^{0.25} \left( \frac{25}{d_b} \right)^{0.20} \left( \frac{l_b}{d_b} \right)^{0.55} \left( \frac{c_{\text{min}}}{d_b} \right)^{0.25} \left( \frac{c_{\text{min}}}{c_{\text{max}}} \right)^{0.1} + k_m K_{tr}$$

In Equation 1:

- $c_{\text{max}}$ and $c_{\text{min}}$ are the greater and the smaller of one-half the clear spacing between lapped bars ($c_s$), bottom cover ($c_y$) and side cover ($c_x$), respectively (see Fig. 2).
- $K_{tr} = n_t n_{st} A_{st} / (n_b d_b l_b)$ is the “stirrup index of confinement”, with $n_t$ = number of legs of the confining stirrup crossing a potential splitting failure surface; $n_{st}$ is the total number of confining stirrups within the lap length; $A_{st}$ is the cross sectional area of one stirrup leg [mm$^2$]; $n_b$ is the number of anchored bars or pairs of lapped bars.
- $k_m$ is the factor representing the efficiency of confinement from transverse reinforcement, whose values vary between 12 to 0 depending on the distance of the stirrup legs crossing the splitting plane from the lapped bars.

The beams were made with a similar vertical concrete cover ($c_y$), of 28 mm and 30 mm for 16 mm and 20 mm lapped bars, respectively (Fig. 2). As a result, the minimum concrete cover/bar diameter ratio ($c_{\text{min}}/d_b$) varied from 1.04 to 1.75, depending on the spacing between the lapped bars, which increased with a reduction of the percentage of lapped bars. The bars were arranged with the ribs oriented in vertical direction. In specimens with 33% or 50% of lapped bars, a 20% reduction of the lap length was applied according to Eurocode 2 provisions, while a reduction of 40% was applied when the percentage of bars lapped at a section was 25%. It should be also noted that the reduction in lap length ($l_b$) was adopted by considering the expected benefits to confinement provided by FRC; in fact, the latter limits the splitting crack propagation due to the bursting wedge action of ribs, thus allowing a more uniform distribution of bond stress over the lap-length.

Each beam was labeled with the bar diameter first; the subsequent characters indicates the type of bars (C stands for continuous bars in the reference beam and L refers to lapped bars); the next number refers to the fraction of bars lapped at the section, followed by the position of the splices (I: internal; E: external) and the lap length (0.8$l_b$ or 0.6$l_b$) with respect to the full lap-length ($l_b$) of the beam when 100% of bars are lapped (Figure 2). The last letters identify plain concrete (PC) or fiber reinforced concrete (FRC).
To comply with Model Code 2010 provisions, the total amount of the transverse reinforcement \((A_{\text{tr}})\) crossing the potential side splitting surface (Fig. 2) was given as a percentage of the area \((A_s)\) of the bars lapped (50% of the total cross-sectional area of all bars lapped at the section). Therefore, two stirrups with a diameter of 10 mm and 8 mm were provided for each pair of 20 mm and 16 mm lapped bars, respectively. These stirrups were evenly spaced along the lap length, with the distance of the first link to the lap end shorter than 50 mm, as required by MC2010\(^{23}\). The shear failure of the beams was prevented by additional stirrups with a diameter of 10 mm placed at 100 mm spacing throughout the shear span. In the present experimental program, the “stirrup index of confinement” \((K_{tr}; \text{Model Code 2010})\), associated with the transversal steel reinforcement only, ranged between 3.1% and 5.0%; the higher values are associated with shorter lap lengths. It should be noted that this index does not take into account the confining effect of steel fibers, whose contribution is provided through FRC post-cracking tensile strength. Further details of the tested beams can be found in Metelli et al.\(^{26}\).

Steel grade B500C was used for longitudinal reinforcements, whose measured mean yield strength \((f_{y,m,\text{ex}})\) was 545 MPa and 523 MPa for 16 and 20 mm diameter rebars, respectively. The relative rib area averaged 0.060 and 0.086 for 16 and 20 mm bar diameters, respectively. The same mix design was used for the two concrete mixtures, the reference plain concrete and FRC, both with a target concrete class of C20/25. The concrete was supplied by a local-ready-mix company. Plain concrete had a slump of 220 mm while FRC had a slump of 170 mm (because of the addition of fibers).
Hooked-end steel fibers ($L_f = 35\, \text{mm}$, $d_f = 0.75\, \text{mm}$ with a tensile strength of 1200 MPa) were used with a volume fraction of 0.5% (corresponding to about 40 kg/m$^3$).

Ten standard 150 mm cube specimens and five cylinders were used for each mixture to evaluate the concrete’s compressive strength and the modulus of elasticity, respectively, for both plain and FRC. The cubes were cast and cured under laboratory conditions until testing the beams; At the time of the tests of the beams, the plain concrete specimens had an average compressive strength ($f_{cm, \text{cube, ex}}$) of about 31.3 MPa, while the compressive strength of the FRC cylinder was 39.8 MPa, which was higher than expected. Finally, FRC can be classified as class “2a” according to fib-Model Code 2010$^{23}$, on the basis of the results of three-point bending tests on beams with notch at mid-span$^{26}$.

2.2 Test set-up

The ten beams were tested with a span of 4.0 m under four-point bending with a constant moment region of 2.0 m, where the lap splice was located$^{26}$. The tests were carried out under displacement control by means of an electro-mechanical screw-jack, up to the splice failure or a mid-span deflection of 80 mm. Linear Variable Differential Transformers (LVDTs) were placed on the bottom surface of the beam to measure the deflections at mid-span and on the supports. Furthermore, two potentiometric transducers with a gauge length of 600 mm were placed on each side of the beam to measure the flexural crack widths within the splice region. Two potentiometric transducers recorded the face splitting cracks developed on the beam's bottom surface. Two additional vertical potentiometric transducers were placed on the sides of the beam at the lap ends to monitor the onset and the opening of the side splitting cracks developing in the rebar plane. Finally, the continuous and lapped bars of the specimens 16L2/4-E-0.8lb-FRC were instrumented with strain gauges, placed 20 mm outside the splice length (in order to avoid any disturbance to the anchorages) for measuring the rebar strain and stress.

3 Test results

As an example, the behavior of the tested beams of series 2 with 16 mm rebars is shown in Figure 3 where the applied load ($P$) is plotted versus the beam deflection ($f$). The slightly non-linear behavior of FRC beams is noted during the transition from the un-cracked to the cracked stage. Initially, several vertical cracks formed within the constant-moment region, followed by shear cracks close to the supports. A typical crack pattern of FRC beams, characterized by more distributed and smaller cracks than in the plain concrete specimens, was observed. FRC enhanced the post-cracking flexural stiffness of the beams, resulting in a reduction of beam deflection at Serviceability Limit State (SLS), ranging between 5 and 25%, for series 1 and 2, respectively. This confirms the capability of FRC to enhance the cracking behavior, because of the beneficial effect of fibers on tension stiffening.

The reference beams (16C or 20C), with continuous rebars, showed a ductile failure due to rebar yielding, whereas the bars splices did not yield when all bars were lapped, which confirms the assumed design criteria of laps. The failure of the beam with lapped bars was governed by the formation of longitudinal splitting cracks along the splices before bar yielding. In PC specimens, both side and face splitting cracks developed, while FRC beams showed mainly diffuse face splitting cracks (on the bottom surface of the beam) with narrow and less evident side splitting cracks which occurred only when all bars were lapped (Figure 4). Face splitting cracks were present in all specimens under each pair of lapped bars, while side splitting cracks were recorded only when all
bars were lapped (both in plain concrete and in FRC), as reported in Table 1. Larger flexural cracks developed at the splice ends, due to the slip of lapped bars at failure.

Unlike the results of previous research on staggered lap splices in plain concrete\(^{25}\), it should be noted, for both series, a higher peak load of the FRC beams with a portion of the bars lapped and a shorter splice length, as compared to PC beams with all bars lapped (Table 1). Furthermore, when all bars were lapped at a section, the splice strength in FRC was 30% greater than in PC beams, due to the enhanced toughness provided by fibers. A volume fraction of fiber of 0.5% seems to be sufficient to provide, through the concrete cover, significant confinement that withstands the bursting wedge action of the ribs bars along the lap-splice; therefore, the post-cracking tensile resistance of FRC is able to limit the splitting crack propagation within the concrete cover. As a result, the bond stresses can be more uniformly distributed along the lap length than in plain concrete; this guarantees a higher lap strength, despite the development of longitudinal splitting cracks. When comparing these results with those of a previous series of tests on PC beams\(^{25}\), it can be noted that the same percentage of transversal reinforcements was not able to arrest the splitting crack propagation.

![Figure 3: Test results: load ($P$) vs. deflection ($f$) of the beams of series 2](image1)

![Figure 4: Typical crack pattern along the lap length on the soffit surface](image2)

The rebar strains measured at the lap ends in the beam with 50\% of bars lapped in FRC beam 16_L2/4_E_0.8lb-FRC are plotted against the beam deflection in Figure 5. In this graph, the load is also plotted versus the beam deflection (thin black line in Fig. 5). On the basis of the bar stress-strain
relationship evaluated by means a uniaxial tensile test on a similar rebar with a diameter of 16 mm, instrumented with strain gauges (see inset in Fig. 5), the tensile stress in the laps and in the continuous bars could be determined. It was observed that, at service loads, the tensile action was evenly distributed among continuous and lapped bars, which showed a tensile stress between 225 and 250 MPa. Moreover, moving to the ultimate load (corresponding to a beam deflection of about 18 mm), the continuous bars started attracting an increasing percentage of the total tensile force in the joint, probably due to the reduced stiffness of the lapped bars that started slipping. This is confirmed by the measurement of the face splitting cracks on the bottom surface of the beam, as discussed below. Therefore, at the ultimate design load, the average bar stress in the continuous bars was slightly greater than in the lapped bars (+6%, as shown in Fig. 5). It can be also observed that the peak load of the beam ($P=112$ kN) was reached when the lapped bars reached the maximum stress of 465 MPa, with the continuous bars having stress close to 500 MPa. The stress in the lapped bar was 14% lower than that expected according to MC2010 expression (without considering the FRC confining contribution; Eq. 1).

The measurements of the strain gauges also provided useful information regarding the post-peak behavior of the lap splices in FRC beams: at a beam deflection of 40 mm, the stress in the lapped bars varied from 367 MPa to 435 MPa. This significant result highlights the relatively stable post-peak behavior of the splices because the FRC post-cracking strength, despite the low volume content of fibers ($V_f=0.5\%$), allowed a better control of the splitting crack development.

Table 1: Summary of the test results

<table>
<thead>
<tr>
<th>specimen</th>
<th>$\mu$</th>
<th>$f_{cm,ex}$</th>
<th>$P_u$</th>
<th>$P_{40}$</th>
<th>$f_{ex}$</th>
<th>$f_{ex}/f_{ym,ex}$</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[%]</td>
<td>[MPa]</td>
<td>[kN]</td>
<td>[kN]</td>
<td>[MPa]</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Series 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>20C-PC</td>
<td>0</td>
<td>28.4</td>
<td>137.8</td>
<td>133</td>
<td>519</td>
<td>0.99</td>
<td>Y</td>
</tr>
<tr>
<td>20L3/3-PC</td>
<td>100</td>
<td>28.4</td>
<td>95.7</td>
<td>48</td>
<td>349</td>
<td>0.67</td>
<td>SP-f-s</td>
</tr>
<tr>
<td>20L3/3-FRC</td>
<td>100</td>
<td>34.5</td>
<td>123.6</td>
<td>77</td>
<td>453</td>
<td>0.87</td>
<td>SP-f-s</td>
</tr>
<tr>
<td>20L1/3-I-0.8lb-FRC</td>
<td>33</td>
<td>34.5</td>
<td>120.0</td>
<td>105</td>
<td>438</td>
<td>0.84</td>
<td>SP-f</td>
</tr>
<tr>
<td>Series 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>16C-PC</td>
<td>0</td>
<td>28.4</td>
<td>127.6</td>
<td>116</td>
<td>550</td>
<td>1.00</td>
<td>Y</td>
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<tr>
<td>16L4/4-PC</td>
<td>100</td>
<td>28.4</td>
<td>66.5</td>
<td>16</td>
<td>274</td>
<td>0.50</td>
<td>SP-f-s</td>
</tr>
<tr>
<td>16L4/4-FRC</td>
<td>100</td>
<td>34.5</td>
<td>85.3</td>
<td>35</td>
<td>353</td>
<td>0.65</td>
<td>SP-f-s</td>
</tr>
<tr>
<td>16L2/4-I-0.8lb-FRC</td>
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<td>34.5</td>
<td>95.4</td>
<td>86</td>
<td>397</td>
<td>0.73</td>
<td>SP-f</td>
</tr>
<tr>
<td>16L2/4-E-0.8lb-FRC</td>
<td>50</td>
<td>34.5</td>
<td>112.1</td>
<td>99</td>
<td>471</td>
<td>0.86</td>
<td>SP-f</td>
</tr>
<tr>
<td>16L1/4-I-0.6lb-FRC</td>
<td>25</td>
<td>34.5</td>
<td>115.4</td>
<td>113</td>
<td>485</td>
<td>0.89</td>
<td>SP-f</td>
</tr>
</tbody>
</table>

$\mu$: percentage of lapped bars; $f_{cm,ex}$: concrete compressive strength; $P_u$: ultimate load; $P_{40}$: post-peak load for a mid-span deflection of 40 mm; $f_{ex}$: average measured bar stress at peak load; $f_{ym,ex}$: measured mean yield strength; Y: bar yielding; SP: splitting failure; f: face splitting crack; s: side splitting crack.
The experimental results of this research program were compared to the results of three well-known semi-empirical models. The first model, proposed by *fib-MC2010*\(^{23}\), includes the effect of most of the variables governing the bond strength, such as concrete strength, bar diameter, lap length, as well as the confinement of cover and transverse reinforcements (see Eq. 1 in Section 2.1). It was calibrated using the results of over 800 tests with ribbed bars in good casting position with a bond index varying between 0.06 and 0.14. This database does not include tests with lap lengths shorter than 15 bar diameters, with low minimum cover smaller than 0.95 \(d_b\), or with concrete compressive strength \(f_c\) lower than 20 MPa.

The semi-empirical expression developed by Zuo and Darwin\(^{27}\) for bottom-cast uncoated ribbed bars also considers the influence of bar geometry by means of the bond index \(f_R\), ranging from 0.069 (conventional bars) up to 0.141 (high ribbed bars). This equation was adopted by the ACI-318 Building Code\(^1\) for design recommendations of bond length. The maximum bar stress from this model can be evaluated according to Eq. 2, where:

\[
\begin{align*}
    c_b &= c_{\min} + 0.5 d_b; \quad \omega = 0.1 c_{\max}/c_{\min} + 0.9; \\
    c_{\min} &= \min(c_i; c_j; c_i/2+6.4\text{mm}); \quad c_{\max} = \max(c_i; c_j; c_i/2+6.4\text{mm}); \\
    K_f &= 6.23 (t_f, t_d, n_t, n_s, f_{cm}^{0.5}) / (n_b, l_b);
\end{align*}
\]

Finally, the model by Harajli\(^{28}\) predicts the bond strength by also taking into account the confinement provided by fiber reinforced concrete cover, as follows:

\[
f_{sm} = 3.16 f_{cm}^{0.25} \left( \frac{c_{\min} + K_c}{d_b} \right)^{2/3} \frac{l_b}{d_b} + 1289 \left( \frac{f_{cm}}{f_{cm}} \right)^{0.25}
\]

where \(K_c\) is the confinement parameter as given by the following expressions:

\[
K_c = K_{df} + K_{cd} = \frac{0.45 c_{\min} V_f L}{d_f} + \frac{7.0 A_{sf} n_t}{s_t n_b}
\]
being $V_f$ the fiber volume fraction, $L/d_f$ the fiber aspect ratio and $s_t$ the spacing between transverse reinforcements.

Figure 6 shows the “bond strength ratio” (BS), defined as the ratio of the maximum average measured bar stress ($f_{s,ex}$) to the estimated bar stress at peak load ($f_{stm}$) using the three previously mentioned semi-empirical expressions. As a first approximation, the bar stress was calculated by means of a cracked sectional analysis at failure and by assuming that the tensile force within the joint is equally distributed between continuous and lapped bars. When referring to the BS ratio, the lap strength of each specimen takes into account the concrete strength, the concrete cover as well as the confinement from stirrups (all these parameters differ among the specimens by varying the percentage of lapped bars and the concrete mixtures). Therefore, the bond strength ratio allows a better evaluation of the effects of FRC and percentage of lapped bars on the strength of the lap splices. Figure 6 shows that the BS ratio of lap splices in FRC, with a reduced lap length ($0.8l_b$ or $0.6l_b$), has a fairly constant trend, independent of the percentage of lapped bars at the section. Furthermore, it should be observed that the measured lap strength was always lower than the one estimated by MC2010 and Harajli expressions ($BS<1$), confirming the results of the former experimental campaign carried out on plain concrete only. Higher Bond Strength ratio was obtained with the Zuo & Darwin model, since it was also calibrated against the bond index of the ribbed bars. On the contrary, it should be observed that the fib MC2010 expression does not consider the effect of the rib geometry, despite the fact that it was based on a database with a wide range of bond indexes of reinforcement ($0.06 \leq f_R \leq 0.14$). This aspect deserves further investigation because, in Europe, bond index values typically range from the required minimum of 0.056 up to 0.09.

The mean face splitting crack width ($w_{fs}$) is plotted against the beam deflection ($f$) in Figure 7 for series 2 with 16 mm diameter bars. It should be noted that the face splitting crack occurred beyond the service condition (with a bar stress $f_c=250$ MPa), thus confirming the low influence of lap splices on the durability. At service condition, larger transversal deformation was recorded in the beam with a single pair of lapped bars with a 40% reduction in lap length (16L_1/4_0.6l_b-FRC) due to higher bursting action associated with greater bond stress. Furthermore, in all other specimens with longer lap length, the addition of steel fibers allowed the face splitting cracks to develop more gradually, as compared to the PC specimen. Smaller face splitting cracks were measured in the beam with external splices (16L_2/4_E_0.8l_b-FRC) with respect to internal splices configuration (16L_2/4_I_0.8l_b-FRC), thus confirming the importance of the position of the stirrups in terms of the splice behavior.
4 Concluding remarks

This research examined the behavior of lap-splices in a FCR, often adopted in practice, with a volume fraction of fibers of 0.5%. In addition to the influence of FRC on splice behavior, the percentage of bars lapped at the section was also investigated. On the basis of the test results, the following outcomes can be drawn:

(i) when all bars were spliced, the addition of fibers led to a significant increase in the lap strength;
(ii) in FRC with part of the bars lapped, no weakening effect was observed in the FRC beam, unlike that observed in similar beams without fiber reinforcements;
(iii) in FRC specimens, the brittleness of the joint was reduced, as compared to the same lap splice configuration in plain concrete, due to the higher post cracking resistance of FRC;
(iv) due to the enhanced confinement of FRC, a reduction of lap length is possible, as compared to specimens without fiber reinforcement.
References


21. ACI Committee 318, 2014, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary, American Concrete Institute, Farmington Hills, Mich.


ASSESSMENT OF FASTENERS TO CONCRETE
A TRIBUTE TO ROLF ELIGEHAUSEN

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ABSTRACT

Some examples are given of assessment of fastenings to concrete structures and the work started by Rolf Eligehausen in fib Task Group 2.9 “Fastenings to structural concrete and masonry”. Studies have been made on e.g. the influence of creep on adhesive anchors and of surface reinforcement and size effects on headed anchors.

1 Introduction

There is often a need to assess the capacity of existing structures. However, the basis for a good assessment is the knowledge of how the structures behave and can be modelled. A large step in direction of establishing this for fasteners was taken when Rolf Eligehausen in 1987 initiated a Task Group on Fastenings to Reinforced Concrete and Masonry Structures.

2 Task Group on Fastenings to Reinforced Concrete and Masonry Structures

The first meeting of the Task Group took place in Stuttgart in November 1987, see Figure 1. It was organized as Task Group VI/5 within Comité Européen du Beton (CEB). The goals of the Group were:

(a) to compile and compare the available research results on the behaviour of fastenings systems

(b) to propose a consistent approach based on current empirical and theoretical models for the design of fastenings

(c) to develop design methods that account for the effects of fastenings and the loads they carry on the behaviour of the structures to which they are attached.

The group met about twice a year and a first state of the art report was published in 1991, CEB (1991)¹,². Then a first guideline was published in 1995, CEB (1995)³. Revised versions of the guideline were published in 1997 and in 2011 by the new organization fib (Fédération internationale du béton – International Federation for Structural Concrete) in fib Bulletin 58 (2011)⁴. The new
organization was a merge between CEB and FIP (Fédération Internationale de la Précontrainte - International Federation for Prestressing). The Task Group has in-between been renamed to Task Group 2.9 “Fastenings to structural concrete and masonry”. A text book was also published, first in German and later in English, Eligehausen et al. (2006)\(^5\). Work is now in progress to make a revision of the guide to include new aspects as assessment of existing anchors.

Figure 1: Photo from first meeting of CEB Task Group on fastenings in Stuttgart, November 4th, 1987. From left: Lennart Elfgren, Johann Tshositsch, Rüdiger Tewes, Klaus Latenser, Werner Fuchs, Kent Gylltoft, Vicky Covert, Rolf Eligehausen, Hans-Dieter Seghezzi, Elizabeth Vintzileou, B Blache, Paul Hollenbach and Harry Wievel.

3 Anchor bolts for foundations

Works for anchor bolts in machine foundations was started in Sweden in 1978, Elfgren et al. (1980, 1982)\(^6,7\). The idea to use adhesives for the bonding was brought up and a study visit was made to Rolf Eligehausen in Stuttgart and to producers of anchors. We then started tests on fatigue and longtime properties, Elfgren et al. (1988)\(^8\)

4 Fracture Mechanics

A way to understand the size effect in anchor bolts was to use the fracture mechanics theory. RILEM had two consecutive Task Groups on this and they arranged round robin tests and analyses of anchors, Elfgren et al. (1989, 1998, 2001)\(^9,10,11\), see Figure 2. In the theory of fracture mechanics, the ratio of the elastic energy to the fracture energy was studied. Based on such studies Eligehausen & Sawade (1989)\(^12\) proposed a formula for the capacity \(F_{max} [N]\) of an anchor to be

\[
F_{max} = 2.1 \cdot \left( E_c G_f \right)^{1/2} \cdot h_v^{3/2}
\]

(1)

where \(E_c\) is the modulus of elasticity of the concrete \([N/m^2]\), \(G_f\) is the fracture energy of the concrete \([Nm/m^2]\) and \(h_v\) is the embedment depth of the anchor \([m]\). Here the exponent of the depth is reduced to 1.5 from the earlier used value of 2, to consider a size effect on the tensile capacity of anchors. The size effect predicts that at ultimate load, the tensile stresses in the concrete averaged over the
fracture surface decreases as the thickness of concrete component increases (Eligehausen et al. 2006). This idea was much spread later, see e.g. Ohlsson (1995), Eligehausen et al. (1998).


5 Assessment of Structures

The assessment of structures is often divided into three phases: Initial, Intermediate and Enhanced and you stop when you get satisfying results. An example of a general procedure is shown in Figure 3, see e.g. Schneider (1994), Schneider & Vrouwenvelder (2017), SB-LRA (2007). ISO 13822 (2010) and Paulsson et al. (2016). When assessing the capacity of anchors in a special structure, e.g. in a power plant applications, there can also be a need to subdivide the phases in three steps as: (1) a global seismic analysis, (2) a local pull-out analysis of an anchor, and (3) an updated global analysis including piping and anchor stiffness.

In methods based on the reliability, the probability $p_f$ is studied for the case that the Load Effect ($E$) is larger than the Resistance ($R$), see e.g. Figure 4. When the curves overlaps and $E > R$ there is a certain risk for failure, see e.g. Schneider (1997), EC Reliability (2005).

The variabilities of the load effect and the resistance have a great effect on the load that can be applied to a structure. If by testing, it can be shown that the variability of the resistance can be narrowed; the load-carrying capacity can be increased considerably. This is an argument for producers and construction companies to keep track of the variability in the capacity of installed anchors by e.g. proof loading procedures.
Figure 3: Flow chart for assessment of existing bridges and other structures. Three phases are identified: Initial, Intermediate and Enhanced depending on the complexity of the questions involved, Schneider (1994)\textsuperscript{16}, SB-LRA(2007)\textsuperscript{18}, Paulsson et al. (2016)\textsuperscript{20}.

Figure 4: Probability variation for Load Effect (E) and Resistance (R), EC Reliability (1990)\textsuperscript{21}
6 Recent work

Long time sustained loading tests on adhesive anchors were started in Sweden in 1981. Two types of adhesive anchors (type A and B) were exposed to various in-service conditions and subjected to sustained tension loads of 15, 30 and 45 kN (i.e. approximately 23, 47 and 70% of their mean ultimate short-time capacities, respectively) over more than 28 years. The experiments were terminated in 2013 and the final results and evaluations reported in Nilforoush et al. (2016). The curves of creep displacement versus time for the tested adhesive anchors are shown in Figure 5. The test results showed that the creep deformation increases by increasing the sustained load level. Results indicate that the tested bonded anchors did not fail indoors when subjected to sustained loads up to 47% of their mean ultimate short-time capacity. However, the long-term performance was substantially impaired outdoors, presumably due to temperature and humidity variations, leading to failure for sustained loads higher than 23% of the anchors’ mean ultimate short-time capacity. Based on the results of long-term experiments, the reliability and suitability of the current testing and approval provisions for qualifying adhesive anchors subjected to sustained tension loads was evaluated and several recommendations were provided (see Nilforoush et al. 2016).

![Figure 5: Creep displacement versus time for M16 adhesive anchors of type A and B exposed to various in-service conditions and different sustained tension load levels (Figures reprinted from Nilforoush et al. 2016).](image)

Work on modelling of the influence of surface reinforcement, member thickness, anchors head size and cracked concrete has recently been carried out in collaboration with Stuttgart. The full descriptions and evaluations of the numerical studies on single cast-in-place headed anchors are given in Nilforoush et al. (2017a, b). Based on these studies, it was found that the tensile breakout capacity of headed anchors increases with increasing member thickness; anchor head size and/or if orthogonal surface reinforcement is present (see Figure 6). Based on the numerical results, the CC method was refined by incorporating three modification factors to account for the influence of anchor head size, member thickness and surface reinforcement.
Nilforoush et al. (2017c)\textsuperscript{25} carried out also supplementary experimental studies to verify the numerical results and evaluate the validity of the proposed refined model. The experimental results showed very good agreements with the numerical results.

The proposed refined model may be used for the design of new cast-in-place headed anchors as well as for the assessment of existing anchors.

Recently, Nilforoush et al. (2017d)\textsuperscript{26} studied the tensile behavior of single cast-in-place headed anchors in plain and steel fibre-reinforced normal- and high strength concrete base materials. The experiments showed an increase of approximately 25-50\% on the tensile breakout capacity when steel fiber is present in concrete.

Figure 6. (a) Load-deflection curves of anchor bolts ($h_{ef} = 200\text{mm}$) in uncracked and pre-cracked plain and reinforced concrete slabs, (b) Load-deflection curves of anchor bolts ($h_{ef} = 200\text{mm}$) with various head sizes in plain concrete members (Figures reprinted from Nilforoush et al. 2017 a, b\textsuperscript{23,24}).

### 7 Summary

Some examples have been given for the assessment of fastenings to concrete structures and on the influence of sustained tension loads on the long-term behaviour of adhesive anchors and on influence of surface reinforcement, anchor head size and embedment depth of headed anchors.

### 8 Acknowledgements

Friendly support from Rolf Eligehausen over many years of cooperation is greatly appreciated. The financial support from the Swedish Council for Building Research, Energiforsk and Elsa and Sven Thysells Stiftelse is thankfully acknowledged as well as cooperation with many other colleagues in the Task Group of Fasteners to Structural Concrete and Masonry.
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ABSTRACT
The paper reflects reminiscences of the author on the time when he visited IWB Universität Stuttgart and was engaged at the introduction of simulation techniques for research and development of anchoring technology. The scope of computer code SBETA which was developed for simulation is described. Basic features of material models based on fracture mechanics are mentioned. Application of numerical simulations include the shear resistance of RC beams loaded by anchors in the tensile zone and the mesh sensitivity study of finite element models. Participation in the round robin analysis organized by RELEM includes a parameter study of 2D anchors.

1 Introduction
Resistance of fastenings to concrete relies on concrete tensile capacity. Due to the complex failure mechanism design of fastenings was typically based on empirical models Universität Stuttgart validated by experimental investigations, and still is. However, advances in failure mechanics of concrete material led to a development of rational theories, applicable to design of fastenings. It was made possible by the world-wide research progress in mechanics of concrete structures, as published in the conference series by IA-FRAMCOS and by professional organizations such as FIB, ACI and RILEM. Among numerous important contributions, for brevity, some noteworthy remarkable findings can be mentioned. Hillerborg et al.(1976) introduced a fictitious crack model for concrete cracking. Bazant and Oh (1983) formulated a crack band theory for concrete fracture. The author’s work on smeared crack approach, Cervenka and Gerstle (1971) introduced the experimentally validated smeared crack model for finite elements prior to the fracture mechanics era, with the tensile strength as the only parameter for a crack opening.

During the eighties of the last century an impact of information technology was accompanied by development of simulation tools for research. The author spent several years at IWB Stuttgart as a visiting professor where he participated in the effort to introduce numerical simulations into research and development of fastenings to concrete. It was the time just before the velvet revolution in his native city of Prague, which then opened doors for intensive academic and personal contacts. Some of the achievements resulting from this cooperation are mentioned in this paper.
2 Computer code SBETA

Computer simulations require a program code capable of serving as a tool for support of diverse research activities. In early development stages we started with a plane stress continuum represented by 2D finite element models, which resulted in a program package SBETA. It offered an entry into a continuum mechanics world and provided a tool for simulation of beams and test specimens. The constitutive model for concrete was based on a damage approach, in which the peak strengths was determined from the bi-axial failure criterion due to Kupfer. The model of concrete in tension was quite advanced, based on fracture mechanics and on the compression field theory (MCFT) of Collins in order to reflect the damage due to cracks.

Figure 1: Constitutive model SBETA, IWB Stuttgart 1989.
The crack opening law was linear and fracture strains were obtained from a crack band model. A variable shear retention factor was used. Two crack models were possible, namely rotated and fixed cracks. Even though the program could not simulate 3D effects typical for anchors, it could be applied to numerous topics, such as RC beams subjected to shear failure. Aside from some top theoretical background theories, the main feature of SBETA was user friendliness. This was appreciated by the colleagues in the institute who used it for research. The code was implemented in MS DOS computer operating system available at that time.

3 Loading by fastenings in tensile zone of RC beams

An effect of fastenings located in tensile zone on shear capacity of beams without shear reinforcement was investigated in Stuttgart experimentally by Reuter and Eligehausen shown in Figure 2. This resulted in regulatory documents for design of anchors in tensile zone

![Figure 2: Experiments of loading through anchors in tensile zone of RC beams, Reuter and Eligehausen (1986).](image)

![Analytical and experimental failure patterns.](image)

![Load-displacement diagrams.](image)

Figure 3: SBETA simulation of experiments on the effect of loading through anchors in tensile zone of RC beams.
Beam tests were simulated by SBETA, Cervenka and Eligehausen\textsuperscript{5}, in order to validate the numerical tool and also to extend the original experimental investigation for wider range of situations not tested originally.

The study had proven the suitability of numerical simulations for research in the field of fastenings. An example from this study in Figure 3 shows the comparison of cracked states obtained by simulation and experiment. The reduction of shear strength expressed by the ratio of ultimate loads $P_t / P_{TA}$ was 0.92 in experiment and 0.9 in simulation (where $P_t$ is for the loads applied on top and $P_{TA}$ for combined loads from the top and from the anchors located in the tensile zone.) The anchor effect was well captured by the simulations.

![Figure 4: Mesh sensitivity of crack simulations by smeared crack model SBETA and Microplane model by Ozbolt\textsuperscript{7}.](image)
SBETA numerical model

Crack patterns for variable embedment depths $d$

Figure 5: Numerical simulation of 2D pull-out tests.

Geometry:
- $d = 50, 150, 450$ mm
- $a = d/2, d, 2d$
- $2x = 3d/10, t = d/10$
- Thickness = 100 mm

Lateral constraint: $K = 0, \infty$

Material properties:
- $f_s = 3 \text{ MPa}$, $f_c = 40 \text{ MPa}$
- $E = 30 \text{ GPa}$, $\nu = 0.2$
- $G_f = 100 \text{ N/m}$

Response diagrams for $d=150$mm, variable span $a$ and two crack models: thick-fixed, thin-rotated.

Load-displacement diagrams for lateral constrains: thick-rigid, thin-free. $a/d=1$.

Figure 6: Load-displacement diagrams obtained by numerical simulations in parameter study.
Mesh sensitivity of numerical models

In numerical simulation a principal question is its objectivity with respect to the discrete nature of finite elements, referred often as mesh sensitivity. Many case studies were performed in order to investigate this question and some results were published at FRAMCOS-2 conference by Cervenka\textsuperscript{7}. Examples of the crack patterns reflecting the strain localization for various mesh sizes and orientation are shown in Figure 4. The study had proven that the mesh size effect is significantly reduced for sufficiently small mesh sizes. It was also shown that the nonlocal concept of the microplane model is more objective as compared to the crack band model. In the following years the microplane model was further developed by Ozbolt and successfully used for anchor simulations.

Pull-out test in round robin analysis by RILEM

The use of fracture mechanics for assessment of anchor resistance was a subject the international cooperation investigated by RILEM Committee on Fracture Mechanics. A round robin analysis of pull-out tests was organized with broad international participation. It was aimed for simulations of anchor behavior based on fracture mechanics and finite element models. The contribution performed by SBETA is illustrated in Figure 5. In this case a 2D anchor is considered, the anchor represents a linear fastening object, such as a steel profile and a plane stress state is assumed for the concrete and for the anchor. Variable parameters were: embedment depth \(d\), support span \(a\). Test specimen are described in Figure 5, left. In Figure 5, right are shown failure crack patterns obtained by SBETA simulations for \(d=150\text{mm}\) and spans \(a=50, 150\), and \(450\text{mm}\). The study provided the load-displacement diagrams for variable span/depth ratios and three sizes \(d\). Two types of lateral constrains were considered, \(K=0\) and rigid. Load-displacement diagrams for the study are summarized in Figure 6.

The RILEM study included also some experimental data for limited number of cases, which served for validation of simulations.

Concluding remarks and acknowledgement

The research stay in IWB Stuttgart University in eighties provided to the author a unique opportunity to conduct research, to develop theories and program codes for simulation of concrete structures. First of all he would like to acknowledge the cooperation with Professor Rolf Eligehausen, with whom he developed close professional as well as personal contacts and with the Professor Rehm, department head at that time. The working environment at the institute was efficient and productive as well as relaxed. Cooperation and friendship with Dr. Jozko Ozbolt, our colleague in simulation group was always close and us well remembered. Dr. Radomir Pukl, the colleague from Klokner Institute in Prague performed as great specialist in software and hardware and created graphic tools. Stuttgart University at large was, and certainly is, the world top place to conduct research. It would be a long list to mention all professors, students and other colleagues who helped along.

It seems that historical development confirmed benefits of simulation combined with testing. The endless discussions about interpretations of experimentally observed features can be brought on the rational level and supported by sound arguments. The present practice indicates increased use of
the rational level and supported by sound arguments. The present practice indicates increased use of simulations in R&D of fastenings, as described in the paper presented by Barraclough¹¹ at this symposium.

The above story is by no means an exhausting report about the history of numerical analysis in IWB Stuttgart. Rather than that, it presents reminiscences and glimpses remembered by the author.

Following the success of SBETA the next generation code ATENA⁹ was developed by the author on commercial basis. While it exploits the experiences from Stuttgart group, it is based on top gears of modern IT technology and obviously led by motivated young generation. The author is recently engaged in application of nonlinear analysis in design of RC structures and related reliability issues, Cervenka¹⁰.

References:


PLENARY LECTURE
FASTENING IN CONCRETE CONSTRUCTION
PAST, PRESENT, FUTURE

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ABSTRACT

Most of the fastening elements in use today were on the market around 1970. However, the information of manufacturers in respect to installation of anchors was very limited and for the design of fastenings almost nonexistent. After several severe accidents with fastenings in Germany, since 1974 the Institut für Bautechnik (IfBt) issued Technical Approvals, which regulated single anchors and groups with 2 anchors ≤ M10 in uncracked concrete.

In order to develop test provisions and assessment criteria for anchors in cracked concrete and to develop a general design method for fastenings in 1982 a research group “Fastenings” was started at the Institut für Werkstoffe im Bauwesen, Universität Stuttgart, which was funded mainly by industry.

A guide for testing and assessing metal anchors in cracked concrete was published in Germany by the DIBt in 1989, which with some modifications was accepted in Europe by EOTA in 1997 and in the USA by ACI in 2000. Bonded anchors were added by EOTA in 2002 and by ACI in 2007. Based on research at different universities in Europe and the USA EOTA published ETAG 001 Appendix E in 2013 which is valid for anchors intended to take up seismic loads. In 2016 ETAG 001 for metal anchors was transferred in an European Assessment Document (EAD) and ETAG 001, Annex E in an EOTA Technical Report.

In 1995 the Concrete Capacity (CC-) Method was published, which allows the design of fastenings with metal anchors under arbitrary loads in cracked and uncracked concrete and predicts the real behavior with sufficient accuracy. It was accepted by EOTA in 1997 and by ACI in 2000. The design guide was extended to anchor channels and bonded anchors in Europe in 2009 and in the USA in 2011.

While the testing and assessing of anchors and the design of fastenings has been harmonized almost worldwide, there are still many problems to be solved. One of the most important is the harmonization of design provisions for connections with bonded anchors according to provisions in fastening technique or with cast-in or post-installed reinforcement according to codes for reinforced concrete. Furthermore, the training of installer should be improved significantly.
1 Introduction

The task to connect building components is as old as building construction itself. Throughout history, the job has been handled in different ways depending on the building material, the structural system and the particular requirements of the construction. In concrete and reinforced concrete construction, fastening elements are either secured inside the formwork and cast into the concrete (cast-in place installation), installed in holes drilled into hardened concrete or directly driven into hardened concrete (post-installed installation).

In this paper the development of fastening elements for use in concrete (cast-in and drill installations) since about 1900 in Germany is described, the procedures for testing anchors and the methods for designing fastenings in Europe and the USA are reviewed and mayor open topics are discussed. Sections 2 to 4 are mainly based on Eligehausen, Mallée, Silva

2 Development of fastening elements in Germany since about 1900

Concrete and reinforced concrete construction initially borrowed fastening techniques from other building trades, either unchanged or slightly modified. Trapezoidal wooden strips placed in the formwork were anchored in the concrete via predriven nails and served as an attachment point. (Fig. 1a). Only small loads could be transferred. Since 1916 anchor channels are used consisting of steel channels which are attached to the form work (Fig. 1b). On site steel metal strips must be passed through the back of the channel and bent to transfer tension loads into the concrete. After removal of the formwork components are fixed to the channel by special T-headed bolts. The anchorage may become effective only after a certain slip of the channel to engage the metal strips and it cannot be guaranteed that the metal strips are bent properly on site. To overcome this disadvantage, since the 1970s T- or I-shaped profiles or headed anchors are rigidly connected to the channel back by welding or forging (Fig. 2).

The anchor channel described above may only be loaded by tension loads and shear loads perpendicular to the longitudinal axis of the channel. To transfer shear loads along the length of the channel special channels with serrated lips in connection with matching serrated T-headed bolts (Fig. 3a)) are used since the 1980s. Notching channel bolts creating a notch in the smooth channel lips (Fig. 3b)) are used since about 1990. To ensure an interlocking connection between bolt and channel, the bolts have to be prestressed with a defined torque.

Since the 1970s headed stud anchorages consisting of a steel plate with headed studs welded on are used (Fig. 4). In the beginning, smooth headed studs were employed, later deformed reinforcing bars were used as well. The welding is usually carried out under controlled factory conditions.

To install post-installed anchors holes must be made in the hardened concrete. While in the beginning this was done with a chisel, later this was performed with the help of a drilling machine. The first electrical percussion drilling machine for concrete was developed by Robert Bosch in 1917, which was later much improved also by other companies. Percussion drilling machines exhibit a small impact energy and need a high contact pressure. Much more effective are hammer drilling machines, which exhibit a high impact energy and require a small contact pressure only. The first hammer drilling machine was developed by company Bosch in 1932. It was much improved by
company Hilti in 1967. Today these drilling machines are produced by many companies and used worldwide.

![Fig. 1: Early cast-in systems]{fig1}
- a) Trapezoidal wooded strip
- b) Anchor channel with steel metal strips

![Fig. 2: Anchor channels with anchors]{fig2}
- rigidly connected to the channel back

![Fig. 3: Anchor channels for transfer of shear loads in direction of the longitudinal channel axis]{fig3}
- a) Channel with serrated lips and matching locking channel bolt
- b) Notching channel bolt creating notches in smooth channel lips

![Fig. 4: Fastening with headed studs welded to base plate]{fig4}

In the beginning stone bolts were bonded in a trapezoidal hole with a cement mortar (Fig. 5a)). Also wooded plugs were pressed into a hole and a wood screw was screwed into the plug (Fig. 5b)) to create expansion forces and to fasten the fixture. This type of anchor was quite common until about the 1960s.

To transfer small loads, so-called “Normdübel” were introduced to the market in 1929. It consisted of a hemp rope enclosed by a soft metal strip (Fig. 6a)), in which a wood screw was inserted to create expansion forces. This anchor was replaced by plastic anchors, which were invented by Artur Fischer in 1956 (Fig. 6b)). Today a large variety of plastic anchors is on the market produced by different manufacturers (Fig. 6c)) optimized for different applications. Special plastic anchors for use in cracked concrete are available as well.
Torque controlled expansion anchors were first developed in 1892 (Fig. 7) and much improved for use in uncracked concrete in the 1960s to 1970s (Fig. 8) and for use in cracked concrete since the early 1980s. Deformation controlled expansion anchors (Fig. 9) are available since 1914 (Fig. 9) and further developed in the late 1970s.

While expansion anchors transfer tension loads mainly by friction, undercut anchors (Fig. 10) transfer tension loads by mechanical interlock. Several systems have been on the market since 1970. Because of the beneficial load-transfer mechanism undercut anchors are mainly used in applications with high safety demands (e.g. nuclear power plants).

Bonded anchors using organic binding materials are on the market since 1963 (Fig. 11). At first as capsule anchors and since 1984 as injection anchors. Bonded anchors transfer tension loads via bond stresses. In the beginning unsaturated polyester was used as binding material. The bond strength was relatively low ($\tau_u \approx 10\text{MPa}$) and it degraded with time because of the low resistance of the resin in an alkaline environment such as concrete. In the meantime other binding materials with a high alkali resistance are used and the mean bond strength has been increased to more than 30 MPa. Since 1993 bonded expansion anchors are available, which combine load transfer via bond and follow-up expansion.
Screw anchors (Fig. 12) are on the German market since about 1994. They cut threads into the concrete during installation and transfer tension loads by mechanical interlock. Since 1999 screw anchors are used, which are installed in a hole partly filled with injection mortar, combining load transfer by mechanical interlock and bond.

### Fig. 11: Bonded Anchors

### Fig. 12: Screw anchors (about 1994)

### 3 Development of guidelines for testing of anchors

Most of the anchor types in use today were already on the market around 1970. However, the information of manufacturers for installation and design was very limited: a) No installation torque was defined, b) only mean failure loads of single anchors in uncracked concrete were published, c) no information in respect to edge distance, spacing and concrete strength on failure load was given, d) no design- and safety concept was available.

During the late 1960s and early 1970s several accidents occurred with fastenings in Germany. Several people were killed or injured and significant damage was caused. Therefore in 1971 a working group “Special anchors for structural connections” was formed at the Institut für
Bautechnik (IfBt) (now Deutsches Institut für Bautechnik (DIBt)). The aim was to write a code on structural connections with cast-in or post-installed anchors. Soon it was found out that the knowledge was much too small. Therefore in 1974 the working group, reorganized into a Committee of Experts on “Anchors and anchor channels for structural connections”, agreed in 1975 on test provisions for the assessment of metal expansion anchors in concrete which were mainly based on the work by Weigler. Starting 1975, the IfBt published approvals for expansion-, bonded- and plastic anchors of different manufacturers. Only single anchors with relatively large edge distances and spacings were regulated. The allowable load defined for a certain anchor size was independent of the load direction (tension, shear or combined tension and shear) and was based on the 5%-fractile of measured failure loads using a global safety factor of 3.0. In 1976 the first approval for anchor channels with anchors welded to the channel and since 1978 approvals for groups with 2 expansion anchors ≤ M10 were published. All these approvals were valid for fastenings in uncracked concrete only.

Because of the rather low concrete tensile strength, cracks must be expected in concrete members under service load. This is taken into account in the design of reinforced concrete structures. The crack width under quasi permanent loads must be limited by proper detailing to \( w_{95\%} = 0.3 \text{ mm} \) to 0.4 mm, depending on environmental conditions (Eurocode 2, ACI 318). Experience shows, that these limits are kept in practice (Schießl, Bergmeister). Because reinforced concrete structures are designed under the assumption that concrete is cracked, it is logical to use the same assumption for the design of fastenings. However, the influence of concrete cracking on anchor behavior was hardly known. Therefore, only approvals for redundant fastenings of light ceilings with metal expansion anchors \( F_{\text{adm}} \leq 0.8 \text{ kN} \) were issued since 1976 and for fastenings with single expansion anchors \( h_{ef} \geq 85 \text{ mm}, F_{\text{adm}} \leq 1.5 \text{ kN} \) since 1979. With the issued approvals it was possible to compare the allowable loads of different products because they were based on accepted test programs and assessment criteria. However, they did not cover most of the applications found in practice.

To improve the situation, a research group “Fastening Technique” was founded 1982 at the Institut für Werkstoffe im Bauwesen (IWB) of the Universität Stuttgart under the guidance of Prof. Rehm. The group was mainly funded by industry. Later it became a department at the IWB, which was headed by Prof. Eligehausen until 2009 and by Prof. Hofmann thereafter.

Safe fastenings need suitable fastening elements provided by manufacturers, design by engineers based on reliable design models and correct installation (Fig. 13). The research at the IWB concentrated mainly on the development of test procedures and acceptance criteria to assess the suitability of anchors and to evaluate design values to be used in a relatively “simple” design method which realistically predicts the capacity of fastenings under all conditions. The main focus of the research was on the behavior of anchors in cracked concrete. It was found, that the concrete cone capacity of all anchors is reduced by about 20% to 30% by a crack with a normal width \( w \approx 0.3 \text{ mm} \) compared to uncracked concrete. While in uncracked concrete the concrete tensile stresses are distributed rotation-symmetrically and balanced by hoop stresses, in cracked concrete hoop stresses cannot be transferred across the crack resulting in a low contribution of the concrete parallel to the crack to take up the anchor tension load (Fig. 14).
Fig. 13: Conditions to achieve reliable fastenings and research fields at the IWB

While torque controlled expansion anchors designed for use in cracked concrete work well in uncracked and cracked concrete (Fig. 15), torque controlled expansion anchors designed for use in uncracked concrete may behave unreliably or may not function at all in cracked concrete (Fig. 16). Deformation controlled expansion anchors (e.g. drop-in anchors) were developed for use in uncracked concrete. Their capacity in cracked concrete is only about 50% (full expansion) or 25% (50% expansion) of the value valid for uncracked concrete (Fig. 17). Also the pullout capacity of bonded anchors in cracked concrete is reduced by about 40% to 60% (depending on type of resin) compared to uncracked concrete (Fig. 18). More results are given in Eligehausen et. al.1

Because the behavior of anchors in uncracked and cracked concrete depends on the type and size of anchors, their suitability for the intended use and relevant design properties must be assessed from the results of approval tests. Based on a proposal of the IWB16 in 1989 the DIBt published a guide on test conditions and assessment criteria for approval tests with mechanical anchors for use in cracked concrete17. Corresponding approvals were issued by the DIBt.

Fig. 14: Distribution of forces in the concrete anchorage zone of a headed anchor loaded in tension, after Rehm, et. al.11

Fig. 15: Behavior of torque controlled expansion anchors designed for cracked concrete in uncracked and cracked concrete. Tests by Dieterle et.al.12

Fig. 16: Behavior of torque controlled expansion anchors designed for uncracked concrete in uncracked and cracked concrete. Tests by Dieterle, Opitz13
The test program comprises identification tests of the anchor, suitability tests (behavior under extreme conditions (installation safety, behavior in wide cracks (w = 0.5 mm) and in opening and closing cracks (w = 0.3 mm/0.1 mm)) and tests to establish characteristic resistances to be used in design.

Very similar fastening elements are used worldwide. Therefore in 1997 the DIBt guideline\textsuperscript{17} - with some modifications - was accepted in Europe as ETAG 00\textsuperscript{18}. Part 1 of this guideline is valid for all types of post-installed anchors. Part 2 and Part 3 contain detailed provision for torque controlled expansion- and undercut anchors respectively. Part 4 “Deformation controlled expansion anchors” was added in 1998, Part 5 “Bonded anchors” in 2002 and Part 6 “Anchors for multiple use for non-structural applications” in 2003. Concrete screws were added to Part 3 in 2008. ETAG 00\textsuperscript{18} was valid for predominantly static loading. Based on extensive research\textsuperscript{20-24} performed at different universities over more than one decade, summarized in Mahrenholtz et al.\textsuperscript{25}, Annex E\textsuperscript{26} extended the field of application of ETAG 001 to seismic actions in 2013. Annex E defines two performance categories: C1 for anchors in areas with low seismicity and C2 for anchors in areas with higher seismicity. In 2016 ETAG 001, Parts 1 to Part 4 were transformed into a European Assessment Document (EAD)\textsuperscript{19a} and Annex A as well Annex E into EOTA Technical Report\textsuperscript{19b,19c}. ETAG 001\textsuperscript{18} with Annex A and Annex E\textsuperscript{26} is still valid for bonded anchors.

ACI 355.2\textsuperscript{27}, valid for expansion and undercut anchors, was published in 2001 and ACI 355.4\textsuperscript{28}, valid for bonded anchors, in 2007. Both documents agree very well with ETAG 001\textsuperscript{18}. They are valid for static and seismic loadings. The test provisions and assessment criteria for seismic actions are identical with those in ETAG 001, Annex E\textsuperscript{26} for performance category C1. Currently work is under way to introduce the performance category C2 of Annex E in ACI 355.2 and ACI 355.4.
4 Development of design models

In 1970 no design method for fastenings was available. During the 1970s the κ-method was developed in Germany for fastenings in uncracked concrete and extended to cracked concrete in 1984 (Fig. 19). The allowable load of one anchor of an anchor group with \( \leq 2 \times 2 \) anchors is obtained by multiplying the allowable load of a single anchor with κ-factors (κ < 1), which take into account the influence of edge distances and spacings. The allowable load of a single anchor is the lowest value valid for all load directions and failure modes for the characteristic edge distance \( a_{rk} \) and spacing \( a_k \) given in the approval. It is evaluated from the results of tests. Depending on the anchor system the allowable load in cracked concrete was taken as 0.4- to 0.6-times the value valid for uncracked concrete.

In 1981 Cannon et al.\(^\text{29}\) published the cone model (Fig. 20) to calculate the concrete breakout failure load of fastenings loaded by a centric tension load. It was incorporated in ACI 349\(^\text{30}\). It was assumed, that at peak load the tensile stresses in the concrete are constant over the projected area of the failure cone and equal to the tensile strength. The inclination between failure surface and concrete surface was taken as 45°.

The concrete breakout load of a single tensioned anchor \( N_{u,c}^0 \) is given by Eq. (1).

\[
N_{u,c}^0 = k_1 f_c^{0.5} \cdot k_2 h_{ef}^2 \\
= k' \cdot f_c^{0.5} \cdot h_{ef}^2
\]

(1)

with

- \( k_1 f_c^{0.5} \) = concrete tensile strength
- \( f_c \) = concrete cylinder strength
- \( k_2 h_{ef}^2 \) = projected area of failure cone
- \( h_{ef} \) = embedment depth
- \( k_1, k_2, k' \) constants

For groups or fastenings at an edge the failure load is obtained by multiplying the concrete tensile strength with the actual projected area of the fastening, which is limited by overlapping stress cones (Fig. 20b)) and intersection of cones with concrete edges.

The 45° cone model is a rational mechanical model which for single anchors agrees sufficiently well with the results of tests available at that time (anchors with \( h_{ef} \leq 200 \) mm). However, it neglects the size effect (Bazant\(^\text{31}\)) (Fig. 21) and the assumption of a constant stress over the failure area does not agree with test results (Fig. 22). Therefore, it cannot be extended to anchors with a large embedment depth (see Fig. 23). Furthermore, the assumed characteristic spacing \( s_{cr,N} \) of anchors for full capacity is with \( s_{cr,N} = 2h_{ef} \) smaller than the real value \( s_{cr,N} = 3h_{ef} \), so that the influence of anchor spacing and edge distance on the failure load is underestimated (see Fig. 25). In addition, the calculation of the projected area of overlapping circles is not simple.
Therefore, the Concrete Capacity Method (CC-Method) for the design of fastenings failing by concrete breakout (tension) or concrete edge failure (shear) was developed\textsuperscript{35}. In the US this approach is termed CCD (Concrete Capacity Design) Method. It takes into account the maximum size effect according to linear fracture mechanics which is proportional to \((k_f h^{0.5})\)\textsuperscript{31,32b}. The CC-Method is based on the extended \(\kappa\)-method\textsuperscript{33,34}. It is very user friendly, because it visualizes the \(\kappa\)-factors in the extended \(\kappa\)-method. All load directions and failure modes are taken into account in the design approach. It is described in detail in Eligehausen, Mallée, Silva\textsuperscript{1}.

The mean failure load \(N_{u,c}^0\) of a single tensioned anchor in uncracked concrete is given by Eq. (2).

\[
N_{u,c}^0 = k_1 f_c^{0.5} \cdot k_2 h_{cf}^2 \cdot k_3 / h_{ef}^{0.5}
\]
\[ N_{\text{u,c}}^0 = k \cdot f_{c}^{0.5} \cdot h_{\text{ef}}^{1.5} \]  

with

\[(k_{\text{h}}/h_{\text{ef}}^{0.5}) = \text{size effect on concrete cone failure load}\]

\[ k = 14.6 \ [N^{0.5}/mm^{0.5}] \text{ expansion anchors}\]

\[ = 16.8 \ [N^{0.5}/mm^{0.5}] \text{ headed anchors}\]

In Fig. 23 the results of tests and numerical simulation with single headed anchors are compared with the predictions according to Eq. (1) and Eq. (2). It can be seen, that Eq. (2) is accurate for anchors with \( h_{\text{ef}} < 600 \) mm and conservative for anchors with a much larger embedment depth.

For calculating the failure load of groups the concrete cone is assumed as pyramid (Fig. 24a). The angle of the failure cone is about 35°\textsuperscript{33-35}. Therefore the projected area of the pyramid is a square with a side length \( s_{\text{cr,N}} = 3h_{\text{ef}} \). If the spacing between anchors is \( s \geq s_{\text{cr,N}} \), neighboring anchors do not influence each other (Fig. 24b). If the anchor spacing is \( s < 3h_{\text{ef}} \), the failure cones overlap and the failure load is reduced compared to a group with \( s > s_{\text{cr,N}} \) (Fig. 24c)). Fig. 25 demonstrates that the CC-method predicts the measured failure loads of groups with up to 36 headed anchors with sufficient accuracy, while the 45° cone model is unconservative, because the assumed characteristic spacing \( s_{\text{cr,N}} = 2h_{\text{ef}} \) is too small.

The influence of other parameters (e.g. disturbance of stresses in concrete due to edges, eccentric tension load, moment, concrete cracking) is taken into account by additional factors.

The above described design approach was accepted in Germany in 1993\textsuperscript{37} and approvals based on the CC-Method were issued. In 1995 the CC-Method was accepted by CEB\textsuperscript{38}, in 1997 by EOTA\textsuperscript{39} and in 2000 by ACI\textsuperscript{40}. Based on the research\textsuperscript{41-44} the CC-Method was extended to bonded anchors and the proposed method\textsuperscript{43} was taken over in a CEN Technical Report\textsuperscript{45} in 2009 and ACI 318\textsuperscript{46} in 2011. The behavior of anchor channels under tension and shear load was studied and in three dissertations\textsuperscript{3,47,48}.
and the developed design method\textsuperscript{49}, which is based on the CC-Method, is incorporated in the CEN Technical Report\textsuperscript{45} and the ICC-ES Acceptance Criteria 232\textsuperscript{50}.

The CC-Method and the corresponding test procedures ETAG 001\textsuperscript{18,26}, ACI 355.2\textsuperscript{27} and ACI 355.4\textsuperscript{28} have been accepted with minor modifications in several countries (e.g. China, New Zealand, Australia) and the issued approvals are used in even more countries. Therefore, the testing of anchors and the design of fastenings have been harmonized almost worldwide.

Fig. 24: Influence of anchor spacing of concrete cone failure load of a group with two anchors according to CC-Method\textsuperscript{35}

5 Installation of anchors

To ensure safe fastenings, a correct installation of anchors is required. However experience shows, that installers are not always trained sufficiently well and anchors are not always correctly installed. This is especially true for the installation of bonded anchors. Surveys performed in Germany\textsuperscript{51} and the USA\textsuperscript{52} demonstrated that only about one third of bonded anchors were installed correctly (Fig. 26).

Installation of bonded anchors horizontally or upwardly inclined (overhead installation) should be performed with a device that develops back pressure (e.g. piston plug), because the pressure of the injected adhesive automatically drives the piston plug back out of the hole and thus reduces the likelihood that air will be trapped in the injected adhesive mass\textsuperscript{53,54}. Without such a device a correct installation is difficult in normal holes and almost impossible in large holes\textsuperscript{53,54} (Fig. 27), because the adhesive may run mainly along the injection tube and not or partly only adhere to the wall of the hole thus leaving large voids at the top. When pushing the anchor rod into the hole explosive eruption of air and adhesive may occur. This not only hazardous for the installer but also typically results in a significant residual void at the top of the hole. Therefore, overhead installations without a device to develop back pressure should be discontinued.

According to ACI 318\textsuperscript{46} overhead installation of bonded anchors to support sustained tension load shall only be performed by personnel certified according to a certification program (e.g. the program
developed by ACI/CRSI\textsuperscript{55}) which includes written and performance tests. The tests should be taken by an independent examiner. The author is of the opinion, that installer certification is needed worldwide.

Fig. 26: Cleaning of holes for bonded anchors according to different surveys. Required cleaning method: Blowing and brushing a) Germany\textsuperscript{51}, b) USA\textsuperscript{52}

Fig. 27: Photo of test specimen after the test\textsuperscript{53}. Injection over 2/3 length of a 635 mm long plastic tube (inner diameter 35 mm) with end-cap method

6 Future Tasks

In the following some open problems that should be solved in the future are discussed.

6.1 Extension of field of application

Fig. 28 shows the field of application given in current design guides\textsuperscript{45,56,57}. Fig. 28a) is valid for fastenings without and with normal hole clearance for all load directions and all edge distances, and for fastenings with normal hole clearance close to an edge if loaded by tension loads only. If fastenings with normal hole clearance are located close to an edge and loaded in shear or combined tension and shear the configurations according to Fig. 28b) are covered.

In practice, larger groups close or far away from edges occur and other shapes (round, triangular) are used. Therefore, the field of application of the design provisions should be extended. More research is needed e.g. for round fastenings in case of concrete cone failure (tension load) or concrete edge failure (shear load) (Fig. 29).

The current provisions for testing and assessing of anchors\textsuperscript{18,27,28} and the provisions for the design of fastenings\textsuperscript{39,45,46,56,57} are valid for structural normal weight concrete without fibers. Only ACI 355.2\textsuperscript{27}, ACI 355.4\textsuperscript{28} and ACI 318\textsuperscript{46} has been extended to structural light weight concrete. In general, the
concrete compression strength to be used in design is limited to $f_c = 50$ MPa. Concrete with a compression strength much larger than 50 MPa or high performance concrete with steel or textile fibers are increasingly used in practice. Research is needed to extend the validity of the current provisions for testing and assessing of anchors and design of fastenings to these materials.

![Fig. 28: Field of application covered by fib design guide and FprEN-1992-4](image)

![Fig. 29: Concrete edge failure of a round fastening loaded in shear, Sharma](image)

### 6.2 Fastenings with supplementary reinforcement

Failure load and deformation capacity of a fastening can be significantly improved by supplementary (hanger) reinforcement (Fig. 30). Fastenings with supplementary reinforcement to take up tension and/or shear loads are covered by current design guides. The design provisions are based on strut- and-tie models (Fig. 31). However, they are rather conservative (Fig. 32a)). This is mainly due to the fact, that the supplementary reinforcement must be anchored in the breakout body according to provisions for tension reinforcement and not for shear reinforcement (stirrups). Because the failure mode “strut failure” is neglected, the design provisions are limited to bars with $d_s = 16$ mm.

Based on the work performed by Schmid and Berger and additional testing an improved model has been developed by Sharma et al. for fastenings with supplementary reinforcement (stirrups) under tension and shear loads, which is sufficiently accurate (Fig. 32b)). A detailed description of the model for shear loaded fastenings at an edge is given in literature. The main features of this model are

- The contribution of concrete to carry a part of the applied tension or shear load is taken into account.
- The capacity of hook and straight length to transfer forces into the concrete are calculated separately based on realistic models.
- The activation of stirrups further away from the outer anchors after yielding of neighboring stirrups closer to these anchors is taken into account.
- Strut failure is considered.
However, several problems should still be solved.

— Currently, for fastenings at the edge with several anchors or anchor rows perpendicular to the edge loaded in shear towards the edge it is assumed, that only the back anchor(s) take up the applied shear load if the failure crack is assumed to start at the back anchor(s)\textsuperscript{45,56,57}. This approach is very conservative\textsuperscript{64}.

— The models for strut failure should be improved, in particular for shear loading\textsuperscript{64}.

— In case of fastenings with supplementary reinforcement to take up tension or shear loads only, an interaction equation is used with an exponent $\alpha = 2/3$\textsuperscript{45,56,57}. This approach is too conservative\textsuperscript{64}.

Fig. 30: Influence of supplementary reinforcement on the behavior of a shear loaded anchor close to an edge. Taken from Eligehausen et.al.\textsuperscript{1}

Fig. 31: Strut-and-tie models for fastenings with supplementary reinforcement according to CEN/TS 1992-4\textsuperscript{45} (tension) and Schmid\textsuperscript{59} (shear)
Fig. 32: Comparison of failure loads measured in tension tests with fastenings with supplementary reinforcement with mean values calculated according to a) CEN/TS 1992-4\textsuperscript{45}, taken from Berger\textsuperscript{60} and b) Sharma et. al.\textsuperscript{63}, taken from Eligehausen et.al.\textsuperscript{65}

6.3 Performance design

Fig. 33a) shows a fastening with rigid baseplate with 3 anchors, either with a perfectly brittle behavior (Fig. 33b)) or a perfectly ductile behavior (Fig. 33c)). The failure load of both anchor types is assumed as equal. The current design provisions\textsuperscript{39,45,46,56,57} are based on anchor strength only and the anchor deformation behavior is neglected. Therefore, the characteristic resistance of the fastening is independent of the anchor type. This is correct, if the fastening is loaded by a monotonic centric load. However, if the baseplate is loaded by an eccentric tension load as shown in Fig. 33a), the failure load of the fastening with anchors showing a perfectly ductile behavior is twice as high compared to a fastening with anchors with a perfectly brittle behavior (Fig. 33d)). If a flexible baseplate is used, the difference is even larger. This difference is neglected in the current approach.

The behavior of steel structures depends on the rotation of the connection between column and foundation. The rotation may be significantly influenced by the performance (pre- and post-peak load-displacement behavior) of the anchors used to connect baseplate and foundation. However, currently the total anchor displacement behavior is not given in approvals, but only the expected displacement at service load. When calculating the rotation behavior of the connection, designers must either neglect the anchor behavior or take assumptions, which may not be realistic. Both approaches may lead to incorrect internal forces of the steel structure.

Also in seismic design, the performance of anchors must be known to realistically predict the safety of seismically loaded fastenings, because earthquakes induce deformations and the resulting load on the fastening depends on the anchor stiffness.

For these reasons, in the future the design of fastenings under monotonic and seismic loading should take into account the anchor performance. To reach this goal, the design provisions must be
extended, and the total load-displacement behavior must be described in approvals based on results of tests according to modified test standards which define how to measure the anchor performance.

## 6.4 Seismic strengthening

Many reinforced concrete structures built before the 1980s, when seismic design provisions were introduced in codes for reinforced concrete structures, may not be earthquake resistant as demonstrated in several earthquakes. These structures need strengthening. There are many methods available (e.g. steel bracing, addition of dampers and/or reinforced concrete elements, external post tensioning, use of prestressed or non prestressed fiber reinforced polymers). In most cases, the additional structural elements must be connected to the original structure by fastenings.

The demands on anchor behavior are high: a) they must transfer (high) tension and/or shear loads. b) they may be situated in cracked concrete with wide cracks and subjected to in-phase load and crack cycling (structural connections) and c) the loaded concrete volume may be limited. These conditions have been taken into account in ETAG 001, Annex E. However, the test guideline should be extended to measure and report the load-displacement behavior including post-peak behavior (compare section 6.4). Furthermore, improved models of connections are needed to realistically predict the seismic behavior of strengthened structures.

![Fig. 33: Influence of anchor load-displacement behavior on failure load of a fastening under eccentric tension loading, Bokor](image)

### 6.4 Harmonization of design provisions for connections in reinforced concrete and fastening technique

Fig. 34a) shows a connection between steel column and foundation by bonded anchors. In Fig. 34b) a reinforced concrete column is connected to the foundation by cast-in or post-installed reinforcing bars. Both connections are loaded by the same load. The connection in Fig. 34a) is designed...
according to provisions valid for fastenings\textsuperscript{45,46,56}, while the connection in Fig. 34b) is designed according to a code for reinforced concrete\textsuperscript{5,46}.

The bond behavior of bonded anchors and cast-in or post-installed deformed reinforcing bars is rather similar. Assuming the same diameter, bond length, bond strength and spacing of bonded anchors and reinforcing bars as well as the same concrete strength of the foundation, the characteristic resistances of the two connections should be identical. However, in many application they will be significantly different\textsuperscript{67}.

This is demonstrated for the connection shown in Fig. 35a). It consists of many straight deformed reinforcing bars welded to a stiff base plate. It was cast into a thick reinforced concrete member without stirrups and loaded by a centric tension force\textsuperscript{68}. The bond length of the straight bars was designed according to design provisions for reinforced concrete (ACI 318\textsuperscript{46}) to ensure ductile behavior. In Fig. 35b) the characteristic resistances of the connection calculated according to Eurocode 2\textsuperscript{7} and CEN/TS 1992-4\textsuperscript{45} are plotted as a function of the number of bars. Similar results are obtained if the connection is designed according to the provisions in ACI 318\textsuperscript{46} for cast-in deformed bars and bonded anchors. The figure is valid for the following assumptions: bar diameter = 16 mm, yield strength = 500 MPa, spacing = 115 mm (≈ 7 \(d_s\)), large edge distance, cracked concrete C25/30. For these conditions the bond length required by Eurocode 2\textsuperscript{7} is \(l_b = 450\) mm (28,1 \(d_s\)). The characteristic bond strength according to Eurocode 2 (\(f_{bd} = 5,8\) MPa) is also taken for the design of the connection with bonded anchors. Furthermore it is assumed that the model in CEN/TS 1992-4\textsuperscript{45} is valid also for the actual conditions (\(l_b > 20\) \(d_s\), \(n > 9\)).
Fig. 35: Connection with straight deformed reinforcing bars subjected to a centric tension load

a) Photo\textsuperscript{68}

b) Comparison of the characteristic resistances $N_{Rd}$ calculated according to Eurocode 2\textsuperscript{7} and CEN/TS 1992-4\textsuperscript{45} as a function of number of bars

In Eurocode 2\textsuperscript{7} it is assumed that neighboring bars do not influence each other, if the spacing is $s_{cr} \geq 7d_s$. With the assumed bond length each bar transfers a tension force equal to the yield force. Therefore the resistance calculated according to Eurocode 2 increases linearly with increasing number of bars. However, according to CEN/TS 1992-4\textsuperscript{45} the characteristic spacing is $s_{cr} \approx 17d_s$ (combined bond and concrete failure) or $3h_{ef} \approx 84d_s$ (concrete cone failure). For this reason, for $n > 2$ bars the resistance of the connections calculated according to CEN/TS 1992-4 is less than the yield resistance, because failure is governed by pullout ($n = 4$) or concrete breakout ($n > 4$). For $n = 9$ bars the characteristic resistance for concrete cone failure is only about 60\% of the yield resistance of the connection. Tests have shown, that the capacity of connections with cast-in deformed can be calculated with reasonable accuracy according to the provisions for bonded anchors\textsuperscript{67}.

The different predictions can mainly be explained by the fact that the provisions for the design of the development length in Eurocode 2 do not take into account the failure mode concrete breakout but assume (not require) that the tension force transferred into the concrete is taken up by stirrups. Furthermore, the safety requirements are different. While in fastening technique a partial safety for concrete failure mode $\gamma_{MC} \geq 1,5$ is required, $\gamma_{MC} \leq 1,15$ is used when calculating the development length\textsuperscript{69}. These explanations are valid also when designing the connection shown in Fig.35a) according to ACI 318\textsuperscript{70}.

The different results when designing connections with cast-in or post-installed deformed bars according to codes for reinforced concrete or with bonded anchors according to provisions of fastening technique are not understandable for design engineers. For this reason a harmonization of the design provision including safety requirements is urgently needed.
7 Summary

Most of the anchor types used today were already on the market around 1970. However, the information of manufacturers in respect to anchor installation and design of fastenings were rather limited. After the failure of several fastenings in Germany with people killed or injured, in 1973 a Committee of Experts “Anchors and anchor channels for structural connections” was formed at the Institut für Bautechnik (IfBt). It agreed on “Test provisions for the assessment of metal anchors in concrete” in 1974. Based on this document, the IfBt published approvals for expansion-, bonded- and plastic anchors for use in uncracked concrete. Only single anchors with relatively large edge distances and spacings were regulated and one allowable load for all load directions was given. Since 1978 approvals for groups with 2 anchors ≤ M10 were published.

In 1982 a research group was formed at the Institut für Werkstoffe im Bauwesen of the Universität Stuttgart funded by industry. The aim was to develop test standards for anchors in cracked concrete and realistic design models. Based on the results of this research in 1989 the Deutsche Institut für Bautechnik (DIBt) accepted a guide “Test conditions and assessment criteria of approval tests with mechanical anchors for use in cracked concrete”.

A modified version of the DIBt document was published 1997 by EOTA as ETAG 001. In 2002 bonded anchors and in 2008 screw anchors were added. Based on extensive research performed at different universities in 2013 the field of application of ETAG 001 was extended to seismic actions by Annex E. Two performance categories are defined: C1 for anchors in areas with low seismicity and C2 for anchors in areas with high seismicity.

The American Concrete Institute published ACI 355.2, valid for expansion and undercut anchors, in 2001 and ACI 355.4, valid for bonded anchors, in 2011. Both documents agree very well with ETAG 001 for static and seismic loading (performance category C1). Currently work is under way to introduce the performance category C2 of ETAG 001 into the ACI documents.

In 1981 the cone model was published to calculate the breakout capacity of fastenings. It was incorporated in ACI 349-85. The cone model agrees with the results of tests with single anchors available at that time (anchors with $h_{ef} \leq 200$ mm). However, it cannot be extrapolated to anchors with a large embedment depth, because it neglects the size effect. Furthermore, the model underestimates the influence of edge distance and spacing on the breakout failure load, because the assumed cone angle is too large. Therefore the CC-(Concrete Capacity) Method was developed which is based on the extended $\kappa$-method developed at the University of Stuttgart. The CC-method visualizes the $\kappa$-factors and is very user friendly. It takes all load directions and failure modes into account and predicts the results of a large number of tests with sufficient accuracy. The CC-method was accepted in Germany in 1995, 1997 in Europe and by ACI in 2000. In 2018 harmonized design provisions will be published in Europe as EN 1992-4.

The standards for testing and assessing anchors and the CC-method for designing fastenings have been accepted by many countries. However, several open problems should be solved in the future. The current field of application should be extended to larger groups, different anchor configurations (e.g. round anchorages) and high strength/ high performance concrete. The existing models for fastenings with supplementary reinforcement should be improved in respect to the number of anchors.
of a fastening at an edge that take up shear load, model for strut failure and for combined tension and shear loads. Currently, the design of fastenings is based on anchor strength. In the future, the design should take into account anchor performance (load-displacement relationship including post-peak behavior). Improved models of connections are needed to realistically predict the performance of strengthened structures. Most important, a harmonization of the design provisions for connections in fastening technique and codes for reinforced concrete is urgently required. Finally the training of installer need to be improved. The author believes that certification of installer by independent institutions should be required worldwide.

8 Acknowledgement

The intensive research in the field of fastening at the Institut für Werkstoffe im Bauwesen, Universität Stuttgart over more than 30 years was possible only due to generous funding by industry; in the beginning by Hilti AG and fischerwerke (both since 1982), later also by Upat (since 1985), Würth (since 1996) and several other companies (e.g. Siemens KWU, Halfen, Kahneisen). The author thanks all companies for their support over so many years.

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