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ARTICLE

New approach for the design shear resistance of headed studs in profiled steel sheeting with ribs transverse to supporting beam

This article presents new equations for the design shear resistance of headed studs in solid concrete slabs and new reduction factors that consider the influence of profiled steel sheeting with ribs transverse to the supporting beam. Comparisons with push-out test results show that the current reduction factor k_t in Eurocode 4 does not take sufficiently into account parameters such as geometry or position of the headed stud in the rib. Therefore, the mean shear test results are overestimated by the current equations. When introducing the new approach for the design resistance of headed studs, the main focus is on the description of the basic procedure for the development of the new approach and on the comparison with values in the existing code. Comparisons with push-out test results from a recent European project have shown the good quality of the new approach.

Keywords headed studs; profiled steel sheeting; through-deck welding; pre-punched sheeting

1 Introduction

1.1 Code rules

In EN 1994-1-1 [1] the design shear resistance of headed studs in composite beams with profiled steel sheeting with ribs transverse to the supporting beam $P_{Rd,Tr}$ is determined by applying a reduction factor k_t to the design shear resistance in a solid concrete slab P_{Rd} (see Eq. (1)).

$$P_{Rd,Tr} = k_t \cdot P_{Rd} \tag{1}$$

In the case of a solid slab, the design shear resistance for headed studs P_{Rd} is the minimum of Eq. (2) for concrete failure and Eq. (3) for steel failure.

$$P_{Rd,c} = \frac{0.29 \cdot \alpha \cdot d^2 \cdot \sqrt{f_{ck} \cdot E_{cm}}}{\gamma_V} \tag{2}$$

$$P_{Rd,s} = \frac{0.8 \cdot f_u \cdot \pi \cdot \frac{d^2}{4}}{\gamma_V} \tag{3}$$

where:

α $= 0.2 \cdot (h_{sc}/d + 1)$ if $3 \le h_{sc}/d < 4$
$= 1$ if $h_{sc}/d \ge 4$

d diameter of shank of headed stud ($16\ \text{mm} \le d \le 25\ \text{mm}$)

f_u specified ultimate tensile strength of material of headed stud, but not greater than 500 N/mm²

f_{ck} characteristic cylinder compressive strength of concrete at age considered with density not less than 1750 kg/m³ ($20\ \text{N/mm}^2 \le f_{ck} \le 60\ \text{N/mm}^2$)

h_{sc} overall nominal height of a headed stud (see Fig. 1)

E_{cm} secant modulus of elasticity of concrete

γ_V partial factor for design shear resistance of a headed stud (recommended value is 1.25)

The reduction factor k_t for considering the influence of the profiled steel sheeting with ribs transverse to the supporting beams is determined by Eq. (4):

$$k_t = \frac{0.7}{\sqrt{n_r}} \cdot \frac{b_0}{h_p} \cdot \left(\frac{h_{sc}}{h_p} - 1\right) \le k_{t,max} \tag{4}$$

where:

n_r number of stud connectors in one rib at a beam intersection (max. $n_r = 2$ allowed in calculation)

h_p height of profiled steel sheeting ≤ 85 mm (see Fig. 1)

h_{sc} overall nominal height of a headed stud (see Fig. 1)

b_0 width of trough to EN 1994 [1], Fig. 6.13, with $b_0 \ge h_p$ (see Fig. 1)

$k_{t,max}$ upper limits for reduction factor k_t to EN 1994 [1] (see Tab. 1)

The resistance of headed studs in a solid slab mainly depends on the material characteristics of concrete and

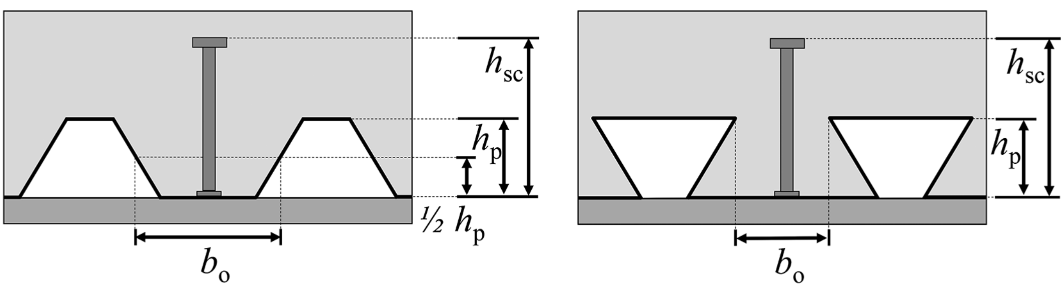

Fig. 1 Composite girders with profiled steel sheeting with ribs transverse to the supporting beam [1]

steel as well as the cross-sectional area of the stud shank (see Eqs. (2) and (3)). Factor α accounts for premature concrete failure of the headed stud if the anchorage length is too short (see, for example, Jenisch [2]). When using Eq. (4), the influence of the trough geometry (b_0/h_p ratio), the embedment depth of the headed stud in the continuous concrete slab above the profiled steel sheeting (h_{sc}/h_p ratio) and the number of headed studs n_r are taken into account (see Tab. 1). Moreover, the upper limit value $k_{t,max}$ (see Eq. (4) and Tab. 1) indirectly takes into account the thickness t of the profiled steel sheeting and whether the headed stud is through-deck welded or the profiled steel sheeting is pre-punched.

Other factors that influence the resistance $P_{Rd,Tr}$ have been neglected. These are, in particular, the position of the headed studs in the trough as well as their relative position to each other (if $n_r \geq 2$). However, a number of studies ([3], [4], [5]) have confirmed the dependence of the headed stud's shear resistance on its position in the trough.

1.2 Discussion of code rules

Test results should be compared with the mean value equations used to derive the design values in the code and not with the design or characteristic values as given in the codes. Eqs. (5) and (6) represent the mean level equations for the shear resistance of headed studs as they are given in Roik et al. [6], which also refer to the mean value of the concrete strength f_{cm}.

$$P_{m,c} = 0.374 \cdot \alpha \cdot d^2 \cdot \sqrt{f_{cm} \cdot E_{cm}} \tag{5}$$

$$P_{m,s} = 1.0 \cdot f_u \cdot \pi \cdot \frac{d^2}{4} \tag{6}$$

Comparing the results for headed studs in a solid concrete slab according to Eqs. (5) and (6) with push-out test results shows very good agreement overall [6]. Hanswille and Porsch [7] subsequently confirmed these results with an enlarged database. The comparison was carried out by determining the statistical mean value, standard deviation and coefficient of variation for the quotient of each test value and the associated calculated value for the database investigated (P_e/P_t). For "concrete failure" (see Eq. (5)) and for "steel failure" (see Eq. (6)), the specific mean value was close to 1.0. Therefore, an improvement to or increase in the design resistance of headed studs in a solid concrete slab in the context of a possible new approach can only be reached through a significant reduction in the scatter in the range of the test results and the associated calculated resistance values.

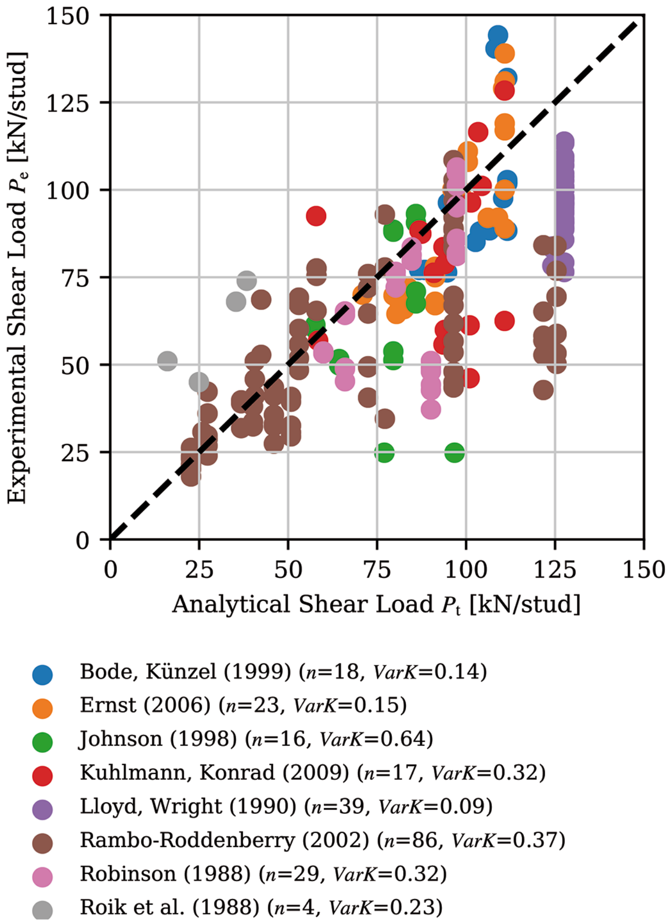

Fig. 2 Comparison of the shear loads from various experimental investigations P_e and the calculated shear resistances P_t considering Eq. (4) and the mean values according to Roik et al. [6], see Eqs. (5) and (6)

However, the evaluation of the code rules for the shear resistance of headed studs for the case of profiled steel sheeting with ribs transverse to the supporting beam using the test database of $n = 300$ in Konrad [10] shows a much smaller correspondence for the mean (see Fig. 2) compared with the values for headed studs in a solid slab.

Tab. 1 Upper limits $k_{t,max}$ for reduction factor k_t according to EN 1994 [1], Table 6.2

Number of stud connectors per rib	Thickness t of sheet [mm]	Studs not exceeding 20 mm in diameter and welded through profiled steel sheeting	Profiled steel sheeting with holes and studs 19 or 22 mm in diameter
1	≤ 1.0	0.85	0.75
	> 1.0	1.00	0.75
2	≤ 1.0	0.70	0.60
	> 1.0	0.80	0.60

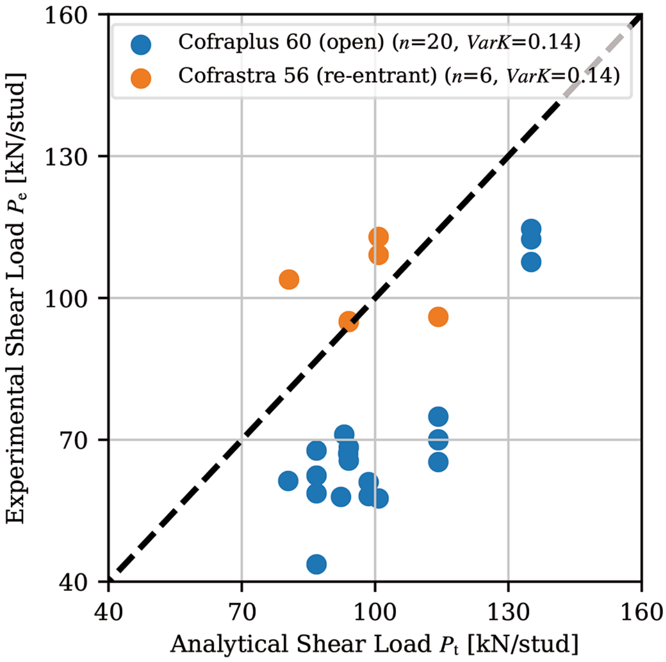

Fig. 3 Load capacity in push-out tests P_e compared with calculated load P_t considering Eq. (4) and the mean values of Eurocode 4 according to Eqs. (5) and (6), as documented in [9]

Recent additional experimental investigations have led to similar findings. Numerous push-out tests using re-entrant steel sheeting (Cofrastra) and open steel sheeting (Cofraplus) with slender ribs were conducted within the scope of the European DISCCO project [8]. Fig. 3 compares the test load P_e with the calculated load P_t using Eq. (4) multiplied by the mean resistance values according to Eqs. (5) and (6) for the solid slab as given in Roik et al. [6]. The test results of headed studs with re-entrant steel sheeting correspond well with Eqs. (5) and (6) according to [6]. This seems plausible as the formulae were developed for this type of sheeting. However, the shear resistance of headed studs combined with open steel sheeting is clearly overestimated, see Fig. 3, indicating that the resistance of this modern profiled steel sheeting shape is not adequately covered by the design resistance given in EN 1994 [1], at least when comparing on a mean level.

On the one hand, this is due to the fact that the positions of the headed studs within the trough are not considered, something that clearly influences their resistance. On the other, EN 1994 [1] does not adequately account for the various geometric parameters influencing the resistance of the headed studs in profiled steel sheeting with ribs transverse to the supporting beam. In addition, Eq. (4) only "pretends" that the trough geometry (b_0/h_p ratio) and embedment depth of the headed stud in the concrete (h_{sc}/h_p ratio) are taken into account. However, when using modern types of profiled steel sheeting, in most cases the upper limit value $k_{t,max}$ becomes critical. Factor $k_{t,max}$ is a quasi-constant reduction factor that depends on the thickness t of the steel sheeting, the number of headed studs per trough ($n_r \leq 2$) and whether the headed stud is through-deck welded or the profiled steel sheeting is prepunched, see Tab. 1.

2 New design equations for the shear resistance of headed stud connectors in solid slabs

2.1 General

A new model that uses reduction factors to determine the shear resistance of headed studs and profiled steel sheeting with ribs transverse to the supporting beams cannot achieve the desired quality without improved equations for the shear resistance of headed stud connectors in solid slabs. Therefore, new equations for the steel and concrete failure of headed stud connectors in solid slabs were developed in Konrad [10] based on the mechanical model by Lungershausen [11]. The model was then validated using a database of n = 140 push-out tests and statistically evaluated according to EN 1990 [12].

2.2 Loadbearing mechanisms of headed studs

2.2.1 Introduction

Lungershausen [18] defined four loadbearing mechanisms for headed stud connectors in solid slabs (see Fig. 4):

A. The compression strut at the welding collar
B. Bending of the shank including the associated vertical force
C. The horizontal component of the normal force in the stud
D. Friction forces between the concrete and steel surfaces

In order to improve the determination of the shear resistance, it is necessary to quantify the different individual loadbearing mechanisms. The frictional forces between the concrete and the beam surfaces (component D) were neglected in the new approach because, on the one hand, there are indications that they do not have a significant influence on the load-carrying capacity [13] and, on the other, they are difficult to quantify in a reliable way.

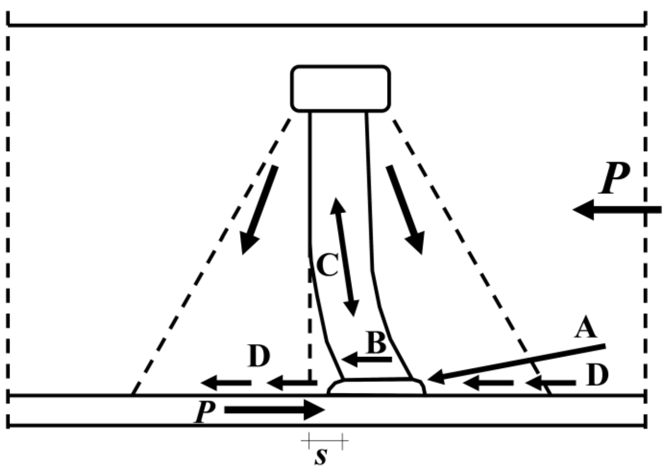

Fig. 4 Loadbearing mechanisms of headed stud connectors in solid slabs [11]

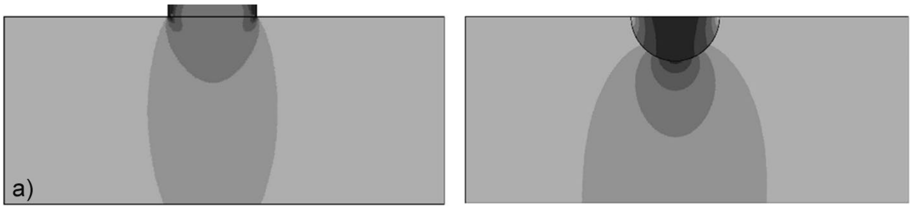

Fig. 5 Elastic stress distribution depending on punch geometry [10]: a) flat punch, b) punch with convex shape

2.2.2 Compression strut at welding collar (A)

For the development of a new approach it is crucial to determine accurately the compressive concrete strength in front of the welding collar or in front of the lower shank area. According to Lieberum [14], compressive stresses develop under a flat punch, as shown in Fig. 5a, with an extreme surface pressure up to 12 to 14 times higher compared with the uniaxial concrete compressive strength.

Furthermore, for punches with a convex shape (welding collar, shank of headed stud, see Fig. 5b), no constant stress distribution can be assumed under the punch. Therefore, the horizontally projected punch area needs to be reduced to 50 %. The compressive force acting on the weld collar (according to mechanism A) can thus be determined by multiplying the reduced horizontally projected welding collar area by the increased concrete compressive strength (modified approach by Lieberum [14]).

2.2.3 Shank bending (B)

The loadbearing contribution due to shank bending is determined on a cantilever as a structural system equivalent to the lower shank of the headed stud (see Fig. 6).

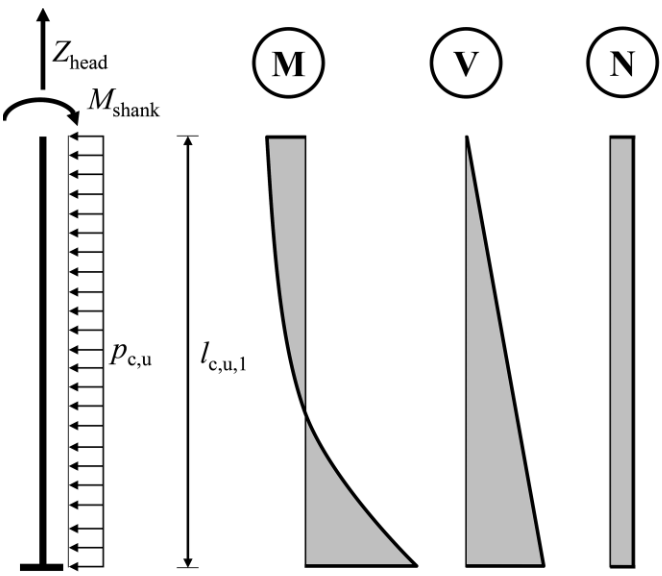

Fig. 6 Cantilever as equivalent structural system for the lower shank for determining the loadbearing contribution due to bending of the stud [10]

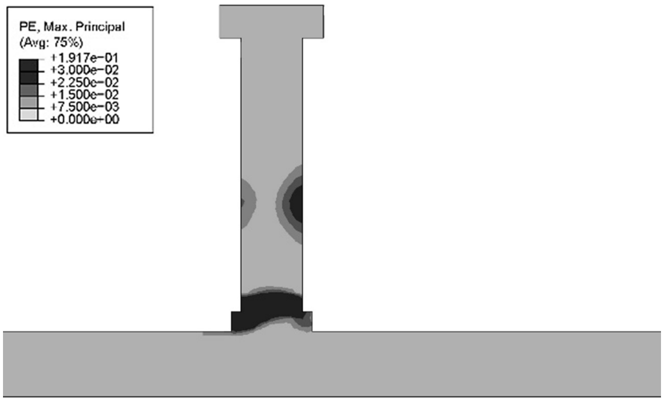

Fig. 7 Plastic strains in a headed stud in a solid slab [10]

For simplification, a constant pressure may be assumed over the shank length. Thus, the relevant shear force is obtained by multiplying the concrete pressure in front of the stud shank $p_{c,u}$ (based on the approach by Lieberum [14], modified by Konrad [10]) by the length of the equivalent structural system $l_{c,u,1}$. This length can be determined through moment equilibrium on the cantilever stud system. As an upper limit, the plastic moment resistance of the shank may be assumed. Numerical investigations showed that, on the one hand, the stud shank is not fully plasticized between the welding collar and the studs head. On the other hand, the full plastic moment cannot be applied here due to interaction with the high shear load at the base of the headed stud (see Fig. 7). Comparative calculations showed that the assumption of 80 % of the plastic moment is reasonable for the base and the shank.

2.2.4 Horizontal component of normal force in stud (C)

The horizontal component of the stud's normal force results from the anchorage of the head in the surrounding concrete and depends on the slip occurring at the base and the axial force in the shank when reaching the maximum load. Although the normal force in the stud Z_{shank} can be estimated as being approx. 35 % of the shear force due to the shank bending in accordance with Zhao [15], the slippage shows a high scatter and is therefore difficult to quantify.

Nevertheless, Konrad [10] showed that a linear concrete compressive strength-slip relationship is sufficiently accurate for determining the deformation at the stud base. The

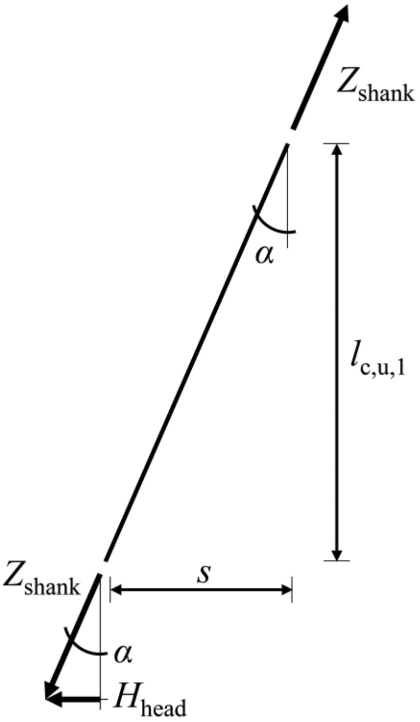

Fig. 8 System for determining the horizontal component H_{head} of the normal force Z_{shank} in the stud according to Konrad [10]

horizontal component of the stud normal force H_{head} thus results from the calculated shear force in the shank according to section 2.2.3 considering the angle α of the deformed structural system (see Fig. 8).

2.3 New design equations for headed stud connectors in solid slabs

2.3.1 Concrete failure

Summing up the individual loadbearing components of the different mechanisms given in section 2.2 and taking into account a few simplifications for applicability leads to a new equation for the shear resistance of headed stud connectors in solid slabs in the case of concrete failure $P_{m,c}$ (see Eq. (7)). For further details, see Konrad [10].

$$P_{m,c} = 39.85 \cdot A_{w,eff} \cdot f_{c,cyl}^{\frac{2}{3}} + 3.75 \cdot d^2 \cdot f_{c,cyl}^{\frac{1}{3}} \cdot f_u^{\frac{1}{2}} \tag{7}$$

where:
$A_{w,eff}$ effective projected weld collar area [mm^2] (see Tab. 2)
$f_{c,cyl}$ uniaxial cylinder concrete compressive strength [N/mm^2]
d shank diameter of headed stud [mm]
f_u tensile strength of stud shank [N/mm^2]

2.3.2 Steel failure

Up until now, the possibility of shearing off the steel shank of the headed stud has not been considered as the critical failure mechanism. Owing to the minimal slip at maximum load, the shear force due to shank bending and the horizontal component of the stud normal force are negligible. Rather, the stud shank almost completely fails due to pure shear. For this failure mode especially, the considerations of Konrad [10] showed a very high quality at the mean value level of the normative approach ($P_{m,s} = A_s \cdot f_u$, where A_s = shank area and f_u = tensile strength of stud material). However, the new approach includes the compressive force on welding collar component, which was determined to be about 25 % of the total shear resistance, see Jenisch [2].

Considering the compressive strut acting on the welding collar as a separate component, the headed stud shear resistance in the case of steel failure $P_{m,s}$ results from adding the component of the strut acting on the welding collar (25 % of total load capacity) to the product of shank cross section A_s and stud tensile strength f_u. Applying a factor of 0.59 leads to a ratio of 75 % of this component with respect to the total shear resistance. This results in Eq. (8):

$$P_{m,s} = 39.85 \cdot A_{w,eff} \cdot f_{c,cyl}^{\frac{2}{3}} + 0.59 \cdot f_u \cdot d^2 \tag{8}$$

where:
$A_{w,eff}$ effective projected weld collar area [mm^2] (see Tab. 2)
$f_{c,cyl}$ uniaxial cylinder concrete compressive strength [N/mm^2]
d shank diameter of headed stud [mm]
f_u tensile strength of stud shank [N/mm^2]

Tab. 2 Effective projected welding collar area according to Konrad [10]

Shank diameter of headed stud [mm]	Height of collar h_{collar} [mm]	Diameter of collar d_{collar} [mm]	Effective projected welding collar area $A_{w,eff}$ [mm^2]
16	4.5	21.0	47.3
19	6.0	23.0	63.0
22	6.0	29.0	87.0
25	7.0	31.0	108.5

2.3.3 Design model for stud resistance in solid slabs

The corresponding design values to Eqs. (7) and (8) were determined according to EN 1990 [12], D.8.2.2 [10]. Variation coefficients for ultimate tensile strength of headed stud f_u, diameter of shank of headed stud d, effective projected welding collar area $A_{w,eff}$ and concrete compressive strength f_c were adopted by [6] and [25]. The high quality of the new design equations is shown by comparison with push-out tests in Fig. 9 for concrete failure and in Fig. 10 for steel failure. In the case of concrete failure, the statistical evaluation according to EN 1990 [12] resulted in a design shear resistance 23 % higher than that given by EN 1994 [1] (see Eq. (9)). On the other hand, for steel failure, the design value is reduced by about 7 % (see Eq. (10)). However, this reduction is based primarily on the much larger scope of application of Eq. (10). Thus, in the statistical evaluation, tests with higher-strength materials ($f_{ck} \leq 100$ N/mm², $f_{uk} \leq 740$ N/mm²) have been taken into account. Owing to the significantly extended scope of application of this approach, this small reduction is justifiable.

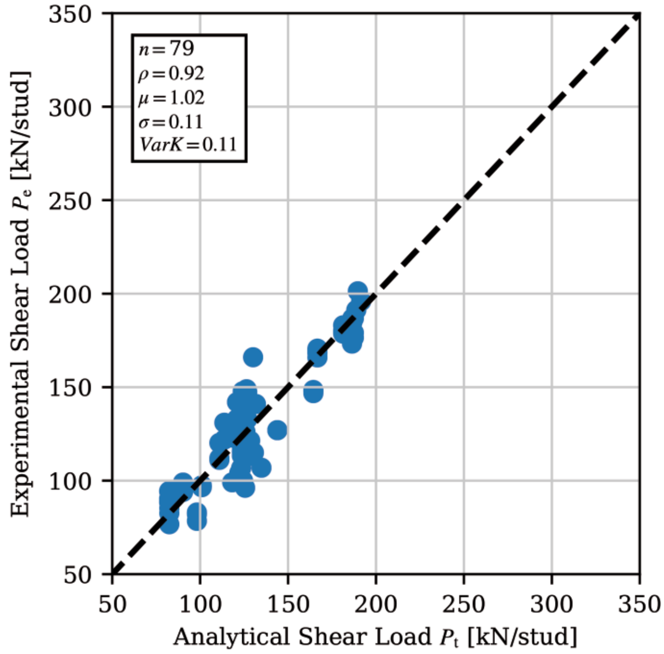

Fig. 9 Concrete failure – comparison of experimental and calculated load capacities according to Eq. (7) [10]

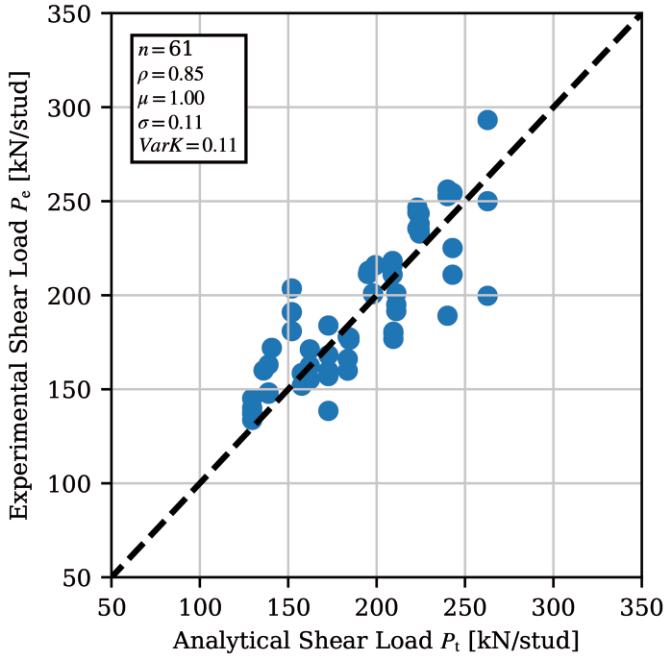

Fig. 10 Steel failure – comparison of experimental and calculated load capacities according to Eq. (8) [10]

$$P_{Rd,c} = \left[\begin{array}{l} 326 \cdot A_{w,eff} \cdot \left(\dfrac{f_{ck}}{30 \dfrac{N}{mm^2}} \right)^{2/3} + \\ 220 \cdot d^2 \cdot \left(\dfrac{f_{ck}}{30 \dfrac{N}{mm^2}} \right)^{2/3} \\ \cdot \left(\dfrac{f_{uk}}{500 \dfrac{N}{mm^2}} \right)^{1/2} \end{array} \right] \cdot \frac{1}{\gamma_V} [N] \qquad (9)$$

$$P_{Rd,s} = \left[\begin{array}{l} 313 \cdot A_{w,eff} \cdot \left(\dfrac{f_{ck}}{30 \dfrac{N}{mm^2}} \right)^{2/3} + \\ 240 \cdot d^2 \cdot \left(\dfrac{f_{uk}}{500 \dfrac{N}{mm^2}} \right) \end{array} \right] \cdot \frac{1}{\gamma_V} [N] \qquad (10)$$

for a headed stud shank diameter 16 mm ≤ d ≤ 25 mm where:

$A_{w,eff}$ effective projected welding collar area [mm²] (see Tab. 2)
f_{ck} uniaxial characteristic concrete compressive strength [N/mm²] (20 N/mm² ≤ f_{ck} ≤ 100 N/mm²)
d shank diameter of headed stud [mm] (16 mm ≤ d ≤ 25 mm)
f_{uk} characteristic tensile strength of stud shank [N/mm²] (f_{uk} ≤ 740 N/mm²)
γ_V partial factor = 1.25

3 Derivation of new reduction factors for shear resistance of headed studs in steel sheeting with ribs transverse to supporting beam

3.1 General

New reduction factors were determined based on the equations of the average shear resistance for headed stud connectors in solid slabs. Qualitatively and quantitatively, these factors take into account the different influencing parameters better than the current rules in EN 1994 [1]. Owing to the large number of parameters, Konrad [10] carried out a comprehensive numerical parametric study. Regression equations were developed from various individual results depending on the position of the headed studs and for the various dependencies. The actual reduction factors were then summarized and weighted through comparisons with push-out tests.

3.2 Push-out tests on headed studs in profiled steel sheeting

A total of 17 push-out tests with different specifications was carried out by Kuhlmann and Konrad [5]. The experimental setup investigated the influences of the position of the headed stud in the rib and the positions of two headed studs relative to each other. This is currently neglected by the code rules [1] and underrepresented in available experiments described in the literature. The influence of the embedment depth and the influence of the lower reinforcement layer on the load capacity were also investigated. Only profiled steel sheeting were used which fulfil the conditions of EN 1994 [1] – ThyssenKrupp T85.1, Cofrastra 70/183 and Holorib HR51/150.

In all experiments, no obvious failure mechanism could be determined upon reaching the maximum load. Fig. 11 shows, as an example, test specimen V1-TK-2f upon reaching the maximum load with a deformation of about 1.5 mm.

Various failure mechanisms that are typical of push-out tests occurred almost simultaneously. Unanswered questions are: Which is the critical mechanism and to what extent can these mechanisms be used to determine the loadbearing capacity? As shown in Fig. 12 for the same test, V1-TK-2f, in the lower rib the headed studs were sheared off and a concrete breakout occurred in front of the stud base. The upper rib also exhibited a concrete breakout that ran out to the concrete slab edge on one side and thus may be interpreted as rib shearing. These experimental observations indicate that the two failure mechanisms, rib shearing and concrete breakout in front of the headed stud, seem to have the same cause: the maximum concrete pressure being exceeded in front of the stud base. Furthermore, owing to the large deformations of the headed studs in the lower rib, a steel failure of the headed stud by shearing off also does not seem to have been the determining factor for the maximum load. In summary, the loadbearing capacity of headed studs in push-out tests with profiled steel sheeting with ribs transverse to the supporting beam in these and many other cases was determined by a concrete failure mechanism, i.e. the maximum concrete pressure being exceeded in front of the stud base. For various design approaches in the literature which consider the different failure mechanisms, the statistical evaluation based on a test database of approx. 300 tests results also showed unsatisfactory results, see [5], [10]. Therefore, a design model that follows the traditional method of reduction factors applied to the stud resistance in a solid slab seemed to be more promising. However, the reduction factors should be improved to take into account the crucial influences.

In summary, 14 out of 17 tests carried out by the leading author did not reach the expected mean shear resistance according to the background to EN 1994 [1], see Eqs. (5) and (6). In particular, the great influence of the stud position on the observed shear resistance is highlighted here. Headed studs in an “unfavourable position” (see Fig. 13) clearly fall below the expected load capacity. On the other hand, results for studs in “mid-position” or a “favourable position” (see Fig. 13) were only slightly below the calculated loadbearing capacity. Thus, the results of the tests conducted by the leading author also confirm the statement that EN 1994 [1] overestimates the loadbearing capacity when using modern profiled steel sheeting with ribs transverse to the supporting beam (see Fig. 2).

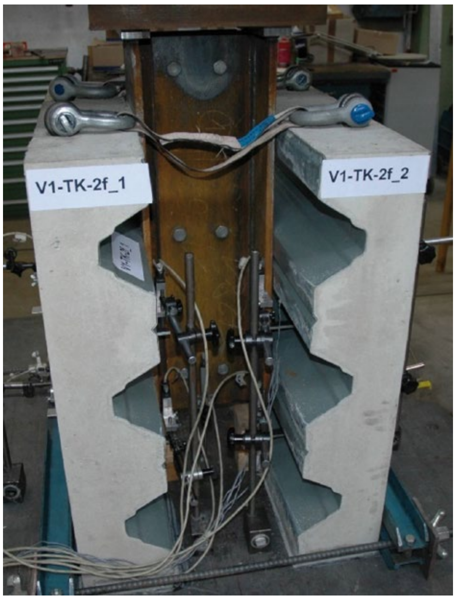

Fig. 11 Specimen V1-TK-2f after reaching maximum load capacity [10] (ThyssenKrupp T85.1 profiled steel sheeting)

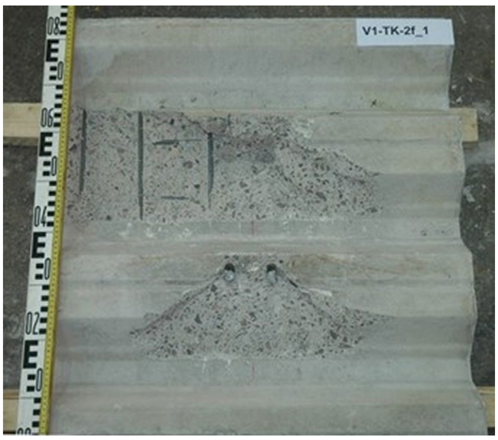

Fig. 12 Inside of concrete slab of specimen V1-TK-2f [10] (ThyssenKrupp T85.1 profiled steel sheeting)

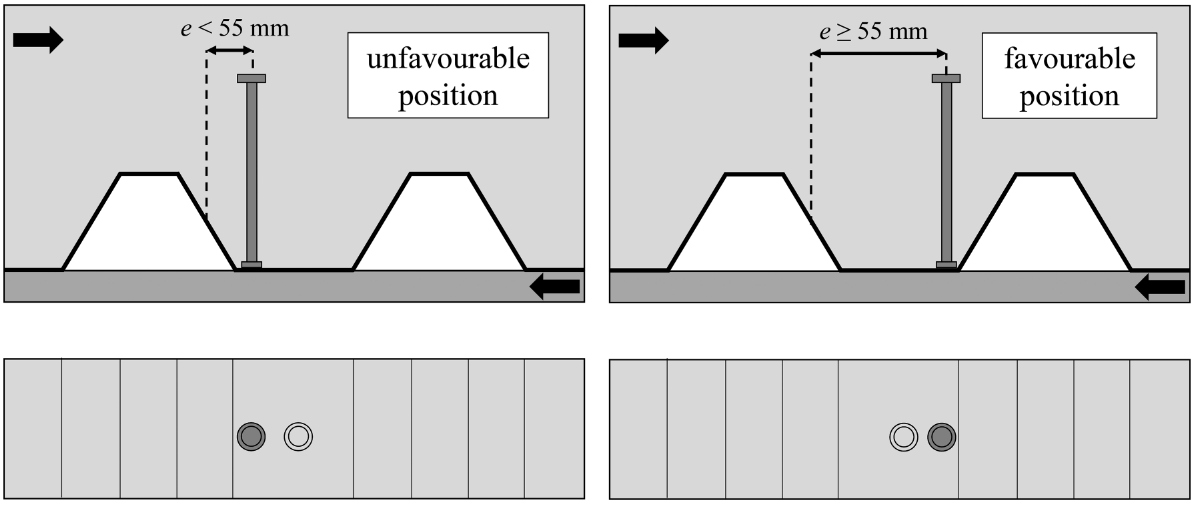

Fig. 13 Positions of headed studs in the trough of the profiled steel sheeting [10]

Another parameter in the push-out tests was the effect of the lower reinforcement layer. In some tests no lower reinforcement layer was embedded in the slab on top of the profiled steel sheeting. In other tests the distance between the lower reinforcement layer and the top of the profiled steel sheeting was varied. The investigations into the influence of the lower reinforcement layer showed that it has no clearly positive effect on the shear resistance, but a positive effect on the deformation behaviour of the headed studs. In contrast to the maximum load, 10 out of 17 of the authors' own tests met the 6 mm ductility criterion according to EN 1994 [1].

3.3 Numerical investigations

To determine the critical parameters influencing the load-bearing behaviour of headed studs in profiled steel sheeting with ribs transverse to the supporting beam, a numerical model was developed in ABAQUS 6.5 [16] which was validated with the tests described in section 3.2. The material properties of the concrete slab were modelled using the "concrete damage plasticity" model. Steel material properties (profiled steel sheeting, headed studs, beam) were modelled with bilinear stress-strain relationships. The symmetry conditions of the push-out tests were an advantage [10]. The maximum load as well as the initial stiffness were reproduced well, as Fig. 14 shows. Further information regarding modelling can be found in Konrad [10].

The validated numerical model was used to investigate the effect of the following parameters on the shear resistance of headed stud connectors in combination with profiled steel sheeting with ribs transverse to the supporting beam:

- Embedment depth in concrete above sheeting
- Number of studs
- Thickness of profiled steel sheeting
- Section geometry of profiled steel sheeting
- Stud position in trough

In the parametric study it was found that the rib height h_p has a direct influence on the load-carrying capacity of a headed stud in the trough as well as the embedment depth, here indirectly expressed as the ratio of the stud height to the height of the steel sheeting h_{sc}/h_p. As the embedment depth increases, so too does the load-carrying capacity, up to a value h_{sc}/h_p = 1.56; a value greater than that does not lead to any further increase.

Considering the construction rules according to EN 1994 [1], section 6.6.5.8, this limit of 1.56 may be compared with the minimum embedment depth of $2d$. This results

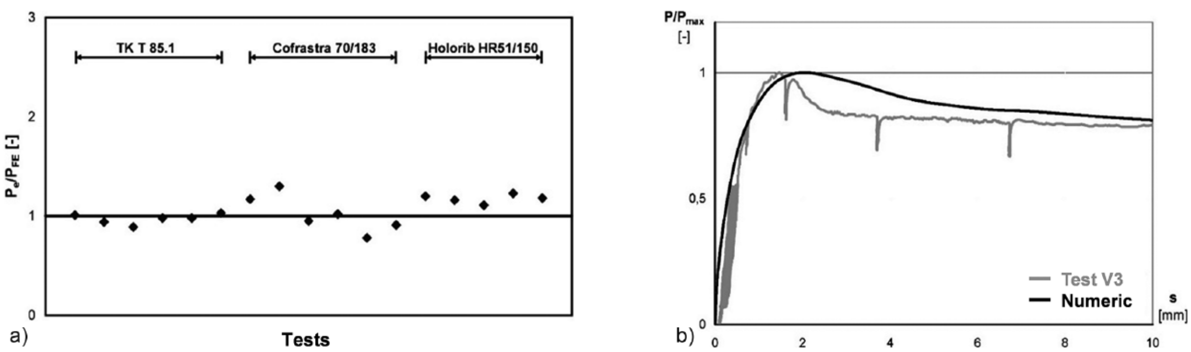

Fig. 14 Validation of numerical model [10]: a) comparison of maximum loads from tests and FE calculations, b) load-slip curves (V3-TK-2u)

for a stud diameter $d = 19$ mm and a constant embedment depth of 38 mm above the top of the steel deck, while the ratio $h_{sc}/h_p \leq 1.56$ for profiled steel sheeting heights h_p from 40 to 80 mm leads to an embedment depth of 22 to 45 mm.

Another important influence is the rib geometry. Only by increasing the b_o/h_p ratio does the position of the headed stud, including the welding collar, play a crucial role. Based on the FE results, the influence of the thickness t of the profiled steel sheeting in the case of open profiles is considered to be small.

It is not easily possible to separate the factors. For example, a change in the rib height h_p with a constant headed stud length h_{sc} always leads to a different embedment depth. Simultaneously, varying the trough width b_o leads to an alternative position of the headed stud in the rib and thus also to a different distance to the web of the profiled steel sheeting.

3.4 New reduction factors for headed studs in slabs with profiled steel sheeting with ribs transverse to the beam

Regression equations were developed from the various individual numerical results depending on the position of the headed stud in the rib (mid-position, favourable and unfavourable, see Fig. 13) considering the parameters that show a high degree of correspondence with the numerical results. The reduction factors were then summarized and weighted through comparison with push-out tests collected in a comprehensive database. The design value of the resistance of headed studs in profiled steel sheeting with ribs transverse to the supporting beam $P_{Rd,Tr}$ is given by Eq. (11). The reduction factors are applied only to the resistance of solid slabs for concrete failure (see Eq. (11)). The resulting k factors are given in the following equations (Eqs. (12) to (14)). A sufficient number of tests was available for an edge distance of the profiled steel sheeting in the load direction $e \geq 55$ mm (see Fig. 15). Therefore, these equations have been statistically verified according to EN 1990 [12] for a partial factor $\gamma_V = 1.25$. For a statistical verification of Eq. (14), more test results than available at the time of [10] are needed to conduct a sufficient statistical analysis according to EN 1990 [12].

$$P_{Rd,Tr} = k \cdot P_{Rd,c} \leq P_{Rd,s} \tag{11}$$

For diameters $d \leq 22$ mm (pre-punched) and 16 mm $\leq d \leq$ 20 mm (through-welded):

1. Pre-punched profiled steel sheeting or through-deck welded headed studs with sheeting thickness $t < 0.75$ mm:

$$k = k_n \cdot \left[k_e \cdot 0.038 \cdot \left(\frac{b_m}{h_p} \right) + 0.597 \right] \leq 1 \tag{12}$$

2. Welded-through headed studs with steel sheeting thickness $t \geq 0.75$ mm and $e \geq 55$ mm:

$$k = k_n \cdot k_{Tr} \cdot \left[k_e \cdot 0.042 \cdot \left(\frac{b_m}{h_p} \right) + 0.663 \right] \leq 1 \tag{13}$$

3. Welded-through headed studs with $e < 55$ mm (simplified, not verified statistically in [10]):

$$k = k_n \cdot \left[0.317 \cdot \left(\frac{b_m}{h_p} \right) + 0.06 \right] \leq 0.8 \tag{14}$$

where:

b_o width of ribs [mm] according to Fig. 15

h_p height of profiled steel sheeting [mm] according to Fig. 15

k_n factor to account for number of headed studs [–]

$$k_n = \begin{cases} 1.00 & \text{if } n_r = 1 \\ 0.80 & \text{if } n_r = 2 \end{cases}$$

k_e factor to account for position of stud in rib

$$k_e = \begin{cases} 1 & \text{if } 55 \leq e \leq 100 \text{ mm} \\ 2 & \text{if } e > 100 \text{ mm} \end{cases}$$

e distance between headed stud and profiled steel sheeting in load direction [mm] (see Fig. 15)

k_{Tr} factor to account for geometry of profiled steel sheeting

$$k_{Tr} = \begin{cases} 1.25 & \text{re – entrant steel sheeting} \\ 1.00 & \text{open steel sheeting} \end{cases}$$

The new approach, as shown in Fig. 16, leads to much better accordance between experiments and calculated values compared with Eqs. (5) and (6), as shown in Fig. 2. This was achieved without limiting the authors' own model by strict application boundaries. Thus, in addition to EN 1994 [1], the new approach requires only a minimum embedment depth of $h_{sc}/h_p \geq 1.56$ (see Fig. 15).

The application limits of the new approach for headed studs in profiled steel sheeting with ribs transverse to the supporting beam were defined because results from push-out tests to check the rules were still lacking. This applies to the sheeting height h_p especially as well as to higher-strength materials. Regarding the sheeting height h_p, only tests with about 100 mm had been available. So far, push-out tests have only been carried out with profiled steel sheeting and cylinder concrete compressive strengths up to about 50 N/mm^2.

4 Verification with modern types of steel sheeting

4.1 General

As part of the European RFCS project "Development of Improved Shear Connection Rules in Composite Beams" (DISCCO) [8], numerous push-out tests were conducted at the Materials-Testing Institute at the University of

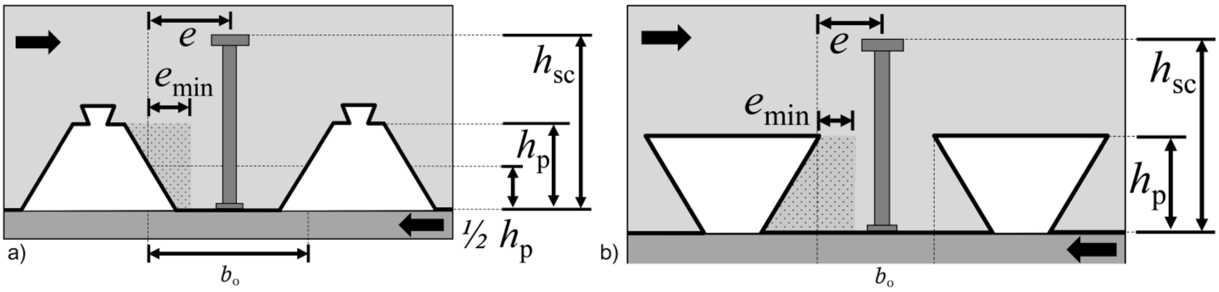

Fig. 15 Geometric dimensions for new reduction factors in Eqs. (12) to (14) according to Konrad [10]: a) open steel sheeting, b) re-entrant steel sheeting

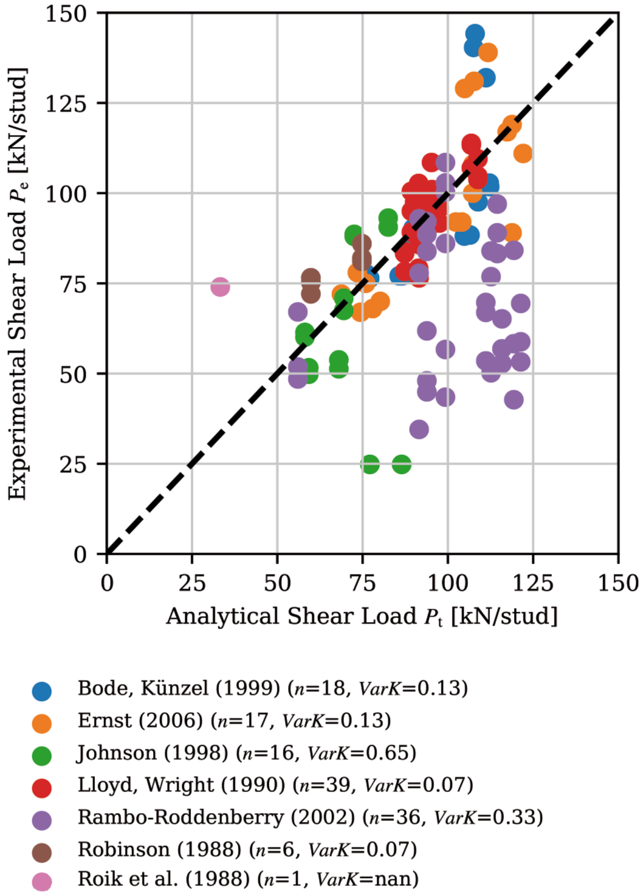

Fig. 16 Comparison of experimental shear resistance P_e and calculated shear resistance P_t using the new approach by Konrad [10]

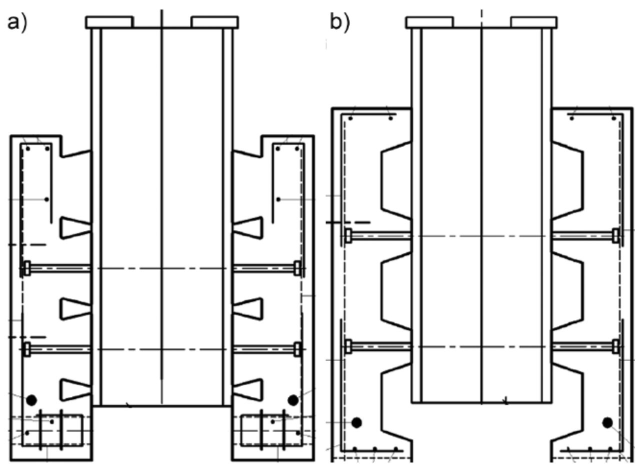

Fig. 17 Push-out test setups [9]: a) Cofrastra 56, b) Cofraplus 60

Stuttgart (MPA). Headed stud connectors were tested with re-entrant and open geometry profiled steel sheeting, both types with ribs transverse to the supporting beam.

In [9] the test results were compared with the equations of the background report to Eurocode 4 given in Eqs. (5) and (6) [6], as shown in Fig. 2. Furthermore, the loadbearing capacities from the tests were compared with the new equations by Konrad [10] presented in sections 2 and 3. Many of these tested combinations of headed studs with profiled steel sheeting are theoretically "excluded" by EN 1994 [1], section 6.6.1.2(3), as they do not fulfil the ductility criteria for a partial shear connection.

4.2 Push-out tests within the DISCCO project

Push-out tests were conducted with an investigation of the following parameters: diameter of shank of headed stud d, number of studs in rib n_r, pre-punched steel sheeting or headed studs welded through and type of profiled steel sheeting. ArcelorMittal's Cofrastra 56 and Cofraplus 60 were used, as shown in Fig. 17.

4.3 Comparing test results with the approach by Konrad and EN 1994-1-1

The shear resistance per headed stud identified in the tests was different from the estimated values according to EN 1994 [1], as shown in Fig. 2.

Fig. 18 compares the shear load from the push-out tests P_e for tests with the re-entrant steel sheeting Cofrastra 56 with the calculated shear resistance based on real geometry and material properties. The current approach and the approach by Konrad [10] correspond well with the test results.

For the open profile Cofraplus 60, the new approach by Konrad [10] corresponds better than the current equations of EN 1994 [1], as shown in Fig. 19. Owing to the geometry of the profile, Eq. (14) was used for an unfavourable position of headed studs in the rib, leading to a slight underestimation of the shear resistance. In contrast, the use of the reduction factor k_t according to EN 1994 [1] leads to a prediction of the shear resistance on the unsafe side according to the tests conducted.

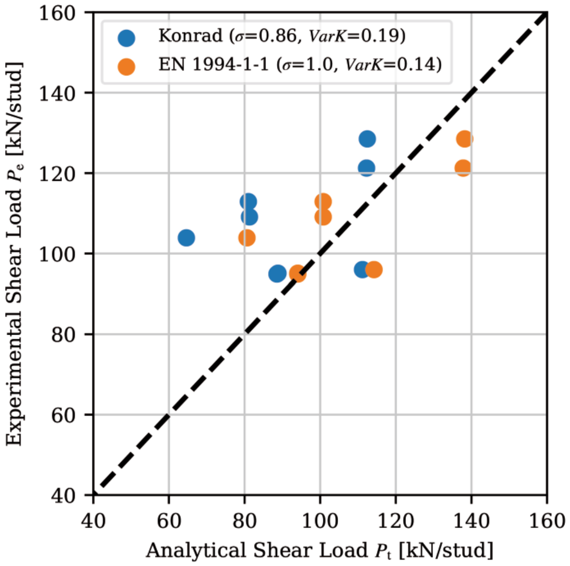

Fig. 18 Cofrastra 56 ($n = 6$) – EN 1994 [1] vs. approach by Konrad [10], see [9], [20]

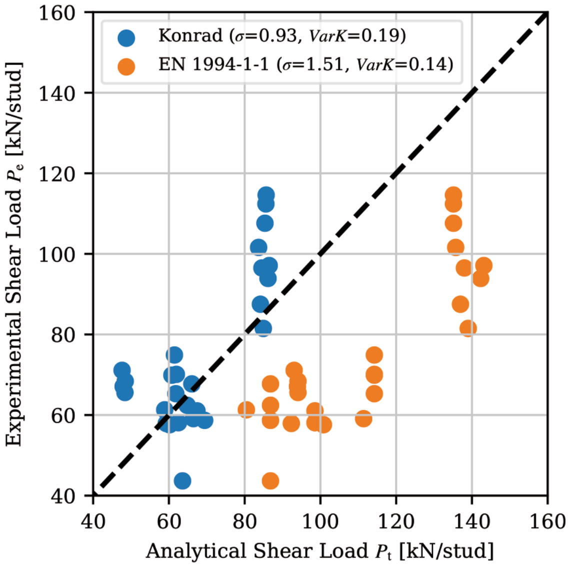

Fig. 19 Cofraplus 60 ($n = 20$) – EN 1994 [1] vs. approach by Konrad [10], see [9], [20]

5 Summary and conclusions

This article has presented a new approach for determining the shear resistance of headed studs in solid slabs and for the use of profiled steel sheeting with ribs transverse to the supporting beam.

The use of the new design equations for headed stud connectors in solid slabs allows a 23 % increase in shear resistance in the case of concrete failure, while the applicable shear resistance for steel failure needs to be reduced by 7 %. However, the approach beneficially allows the use of higher-grade materials ($f_{ck} \leq 100$ N/mm^2, $f_{uk} \leq 740$ N/mm^2) and therefore enlarges the current very restrictive boundary conditions in EN 1994 [1]. This may solve unanswered issues, e.g. load introduction in high-strength composite columns.

The influence of profiled steel sheeting with ribs transverse to the supporting beam was taken into account by new reduction factors that are sufficiently accurate to maintain the required level of safety in contrast to the current standard. The statistical evaluation of the new approach with push-out tests from the literature reaches a quotient-mean of test load to calculated value of 0.99. The standard deviation or coefficient of variation were 0.11. Push-out tests that fulfilled the criteria due to favourable positioning of the headed studs in the rib were included in the statistical evaluation. Push-out tests with a compression force induced by horizontal loading were excluded, likewise tests with an unfavourable position of the headed stud in the rib. The comparison with models from the literature shows that this results in good accuracy, see Konrad [10]. The advantage of this new approach is that it follows the principle of the traditional approach by multiplying the resistance of headed studs in solid slabs by reduction factors, but with improved approaches for both of them.

In a subsequent European research project [8], the good accuracy of the new approach has been validated through comparisons with further test series. In particular, for slender ribs, the number of tests with an unfavourable position was extended and proved to be a successful approach. This includes particularly the reduction factor for $e < 55$ mm (see Eq. (14)).

6 Outlook

Although the k factor achieved good accuracy for headed studs in unfavourable positions ($e < 55$ mm), a statistical evaluation with respect to partial factors according to EN 1990 for this particular case is still underway and will be published in [20].

The k factors were only developed for headed stud connectors in profiled steel sheeting with ribs transverse to the supporting beams. The k factors for applications involving profiled steel sheeting with ribs parallel to the supporting beam need to be checked and, if necessary, modified. For a good check, a sufficient database of additional push-out tests needs to be built up.

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