Rationale of Detailing Provisions in MC 78 and Identification of Missing Items

by

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1. Introduction

As you all know, the quality of a structure depends significantly on the quality of the detailing of the reinforcement. If failures occur, they are often caused by poor detailing, but rarely by incorrect structural analysis. Therefore the MC 78 pays close attention to this subject. The corresponding provisions are given in Section 17 and 18 of the Model Code.

2. General Requirements

When deducing detailing provisions from the results of analytical or experimental investigations, the requirements summarized in Fig. 1 must be taken into account. They are known to you, but let me point out some particularities when dealing with detailing provisions.

In the ultimate limit state, the equation

$$S_d \leq R_d$$

must be fulfilled. With the corresponding partial safety factors $\gamma_g = 1.35$, $\gamma_q = 1.5$ and $\gamma_m = 1.5$ for concrete failure, the total safety factor $\gamma = \gamma_f \cdot \gamma_m$ is 2.0 to 2.25, depending on the ratio dead load to live load. However, different partial safety factors for the actions would unnecessarily complicate the detailing provisions.

Therefore a mean value γ_f = 1.4 was accepted, giving γ = 2.1 against the 5 %-fractile of the available results. In this way it shall be guaranteed that an eventual failure of a structure is not caused by a bond failure.

In the serviceability limit state the critical width of cracks perpendicular to the reinforcement must be smaller than a limiting value. For reinforced concrete the allowable crack width is 0.1 mm to 0.4 mm, depending on the ambient conditions of exposure. To ensure different values for the allowable crack width would significantly complicate the detailing provisions. It was felt, that this was not justified by our present knowledge, e.g. about the influence of the anchorage length on the crack width. Therefore the given detailing provisions aim at keeping the width of cracks in the critical regions smaller than about 0.2 mm to 0.3 mm under dead load and a fraction of the live load. Furthermore,the detailing provisions shall ensure that no splitting cracks, which run almost parallel to the bars, occur under service load.

Summarizing, it can be stated, that
the detailing provisions given in MC 78 aim at ensuring

a sufficient low probability of failure and
an adequate behavior under service load.

However, they are not exactly built up as required by Section 6 of the Model Code, because

a) the strict following of the requirements given in Section 6 of the MC 78 would lead to more complicated detailing provisions.

b) the knowledge about the influence of a certain detail on the behavior of the structure under service load was 1978 and is still incomplete.

The detailing provisions are based on some minimum requirements for cover, spacing and transverse reinforcement, which are summarized in Fig. 2.
These minimum requirements can easily be observed and they make it possible to deduce relatively simple provisions.

3. Rationale of the design provisions for anchorages and splices

3.1 Anchorages

The Model Code allows to anchor bars and welded wire fabric at any location of a structural member by employing different anchorage devices. Because all reinforcement must be anchored, the corresponding provisions should be as easy as possible.

Taking into account the minimum requirements given in Fig. 2, the common places for anchorage shown in Fig. 3 and the fact, that normally only a small part of the total reinforcement is staggered in one section, one may assume, that an evt. bond failure will be caused by pullout rather than by splitting. Therefore the anchorage length of straight and hooked bars and for welded mesh fabric was evaluated from the results of tests where the bond failure was caused by pullout. The criteria used for dimensioning the anchorage length was the slip at the loaded bar end under working and ultimate load for which limiting values were established (Fig. 4).

The validity of this approach is justified by a large number of full-size tests on structural members. It allows for very simple provisions which are summarized in Fig. 5. The basic anchorage length is multiplied by a factor α equal to 1.0 for straight bars. For hooked or looped bars in tension the factor α may be taken as 0.7. For welded wire fabric α is either 0.7 or 0.5, depending on the number of welded transverse wires in the anchorage zone.

Unfortunately, in the Model Code the α -values are not condensed in a table but spread over the different sections.

Fig. 6 shows the ratio between anchorage length and bar diameter as a function of the concrete strength. Compared are the CEB requirements with different proposals given in the literature for a pullout failure. As can be seen, the Code requirements agree rather well with the theoretical values.

3.2 Lapped splices of bars

Experiments have shown that under conditions found in practice lapped splices fail mostly by splitting. Fig. 7 shows three very common failure modes. The failure load can be calculated with sufficient accuracy taking into account all relevant parameters. Fig. 8 shows as an example the measured splice strength as a function of the calculated values using the equations deduced in /3/. The figure is based on 390 tests with and without transverse reinforcement along the splice length from 15 different sources. As can be seen, the predictions agree rather well with the experiments. The coefficient of variation is only 15 %.

In Fig. 9 the lap length as required by the Model Code is compared with the length necessary to ensure a sufficient safety against failure. The latter was calculated using the equations deduced in /3/ and assuming a total factor of safety γ = 2.1 as explained earlier. The assumed values for cover, spacing and transverse reinforcement in the splice region comply with the minimum requirements of the Model Code. Taking into account that code provisions must simplify the real behavior, the lap length required by CEB agrees sufficiently well with the theoretical values.

The Model Code takes only the influence of the percentage of spliced bars to the total reinforcement and of the distance between splices on the required lap length into account. The equally important influences of concrete cover, transverse reinforcement and bar diameter are neglected for reasons of simplicity. This situation should be improved in the next edition of the Code.

3.4 Splices of welded wire fabric

Splices of welded wire fabric are different from splices of bars in the following ways:

- The allowable maximum diameter of the wires is 12 mm.
- Usually the spacing of the wires is 100 mm or 150 mm.
- Welded cross wires are present along the splice length.
- Typically splices are made by layering the fabric above each other in the splice region.
- No reinforcement crosses the plane of failure.

Fig. 10 shows a failure of such a splice. In spite of the relatively large wire spacing, failure usually occurs by splitting off the entire concrete cover, including the fabric nearest to the surface. The failure mode is initiated by the forces normal to the cover due to the eccentricity of the spliced fabric. The splice strength depends - as for splices of bars - mainly on the lap length, the diameter and spacing of the wires and the concrete strength. Of special interest is the influence of welded cross wires in the splice region on the splice strength. It is documented in Fig. 11 which shows the steel stress just outside the splice at splice failure as a function of the number of welded cross wires. As can be seen, the splice strength does not increase much with increasing number of welded cross wires for small spacings of the longitudinal wires and for a lap length as necessary to provide an appropriate splice strength. Therefore the CEB-FIP Model Code does not allow the lap length to vary with the number of welded cross wires.

The lap length l_d/l_{be} as a function of the equivalent wire diameter d_{be} is plotted in Fig. 12. For meshes made from double wires, the equivalent wire diameter is the diameter of a single wire having the same area as the double wire. Each point represents the results of 5 to 12 splice tests. According to the tests, the necessary lap length to ensure the required safety against failure increases significantly with increasing wire diameter.

This is reflected quite well by the CEB Model Code, however, the provisions are somewhat conservative. Splices of fabrics made from double wires with a diameter larger than 8.5 mm are permitted only in the inner layer of a multiple layered reinforcement, to keep the necessary lap length relatively short.

4. Identification of missing items

After discussing some of the detailing provisions of the Model Code, please let me point out some missing items. Three major areas can be identified:

a) Anchorage of prestressed reinforcement
b) Detailing the reinforcement in
   frame corners, corbels, brackets etc.
c) Industrialization of reinforcement

4.1 Anchorage of prestressed reinforcement

The provisions for the anchorage of prestressed reinforcement are relatively brief. For pretensioned prestressed reinforcement the anchorage zone should be designed in the same way as for reinforcing bars. More guidance is also needed for the design of the anchorage zone of posttensioned reinforcement.

4.2 Special cases (frame corners, corbels)

Fig. 13 shows the efficiency of frame corners subjected to positive (opening) moments for different types of corner reinforcements. Plotted is the ratio measured to calculated ultimate bending moment as a function of the reinforcement percentage. It can be seen, that with certain types of corner reinforcement often recommended in literature, the corner will fail before developing the flexural capacity of the adjoining members. No design recommendations are given in the Model Code. The proposal of DIN 1045, which is based on the work of Nillson and Kordina, shown in Fig. 14, ensures an adequate behavior of the corner under service and ultimate load.

Similar to frame corners is the situation in other cases, such as corbels, brackets, inclined girders etc. In these cases detailed provisions are needed for the dimensioning of steel and concrete and the arrangement of the reinforcement. However, these provisions should not be isolated, but they should be based on generally accepted models, which predict the behavior of these members under service and ultimate load with sufficient accuracy. Prof. Schäfer will report about this subject later in the afternoon.

4.3 Industrialization of reinforcements

The need for industrialization of the reinforcement is stressed in Bulletin 164 /8/, which came out last month. The report contains a variety of proposals, which are not covered by the current edition of the Model Code. An example is the shear reinforcement which traditionally consists of stirrups and bent up bars. Much more economical is it to use only straight bars as top and bottom bending reinforcement of continuous beams and to make the shear reinforcement out of open stirrups and supplementary bars, called "shear assemblies" (Fig. 15). The stirrups are closed by the transverse reinforcement of the slab.

The prefabricated reinforcement can be easily assembled in the formwork like a meccano system, as shown in the right part of Fig. 15.

Shear assemblies, which replace bent-up bars, are cage-like, ladder-like or garland-like elements made of deformed bars or deformed welded mesh fabrics which do not or only partly enclose the tension reinforcement (Fig. 16). They can easily be assembled on site (Fig. 17).

Theoretical and experimental studies showed, that the described modified shear reinforcement works as well as a traditional one, the results of the experimental studies are shown in Fig. 18. Plotted is the related shear strength of T-beams reinforced with different types of shear reinforcement. As you can see, the shear strength of T-beams with the proposed shear reinforcement is as high as the shear strength of beams with a traditional shear reinforcement, provided the shear assemblies were well anchored, preferably by welded cross bars.

5. Summary

Let me summarize.
The present Model Code is fairly complete as far as anchorages and splices of ordinary reinforcement is concerned. However, there are still some areas that need substantial improvement.

References

/1/ CEB-FIP Model Code for Concrete Structures, Bulletin D'Information of the CEB, No. 125, Paris, 1978

/2/ Rehm, G., Eligehausen, R. and Neubert, B.: Erläuterung der Bewehrungsrichtlinien von DIN 1045, Deutscher Ausschuß für Stahlbeton, Vol. 300, Berlin, 1979 (in German)

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/4/ Orangun, C.O., Yirsa, J.D. and Breen, J.E.: A Reevaluation of Test Data on Development of Lengths and Splices, ACI-Journal, March 1977

/5/ Tewes, R.: Design of Lapped Splices of Welded Wire Fabric, in: Bartos (Editor) Bond in Concrete, Applied Science Publishers, London, 1982

/6/ Kordina, K.: Bewehrungsführung in Rahmenecken und Rahmenknoten, in: Betontag 1975, Deutscher Beton-Verein, Wiesbaden, 1976 (in German)

/7/ DIN 1045, Beton- und Stahlbeton, Edition Dec. 1978

/8/ CEB TG VII/3: Industrialization of Reinforcement in Reinforced Concrete Structures, Bulletin D'Information No. 164, Lausanne, Jan. 1985

/9/ Rehm, G., Eligehausen, R., Neubert, B. and Lehmann, R.: Rationalisierung der Bewehrungstechnik - Ein unerschöfliches Forschungsthema oder eine Möglichkeit zur Kostensenkung und Qualitätssteigerung im Stahlbetonbau, Bauingenieur 57, Dec. 1982, (in German)

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Ultimate	Serviceability
Limit state	
$\boxed{S_d \le R_d}$ $S_d = \gamma_g \cdot S_g + \gamma_q \cdot S_q$ $R_d = R_k / \gamma_m$ R_k = 5% fractile of resistance $\left.\begin{array}{l}\gamma_g = 1.35\\ \gamma_q = 1.5\\ \gamma_m = 1.5\end{array}\right]$ $\gamma = 2.1$ against R_k	a) Limit of deflections b) $W_{95\%} \leqq W_{lim}$ $W_{lim} \sim 0.25$ mm for $S \cong S_g + 0.5\, S_q$

Fig. 1: Requirements by the CEB/FIP Model Code (1978)

Bar Diameter : $\phi \le 32$ mm (50 mm)

Concrete Cover : $c \begin{array}{l}\ge 15 \text{ mm}\\ \ge \phi\end{array}$

Clear Distance : $s \begin{array}{l}\ge 20 \text{ mm}\\ \ge \phi \text{ (Anchorages)}\\ \ge 2\phi \text{ (Splices)}\end{array}$

Transverse Reinforcement : Required

Fig. 2: General Detailing Requirements according to CEB/FIP Model Code (1978)

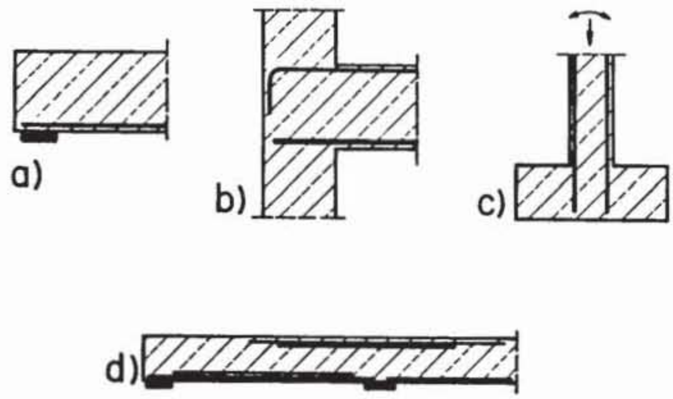

Fig. 3: Common places for anchorages

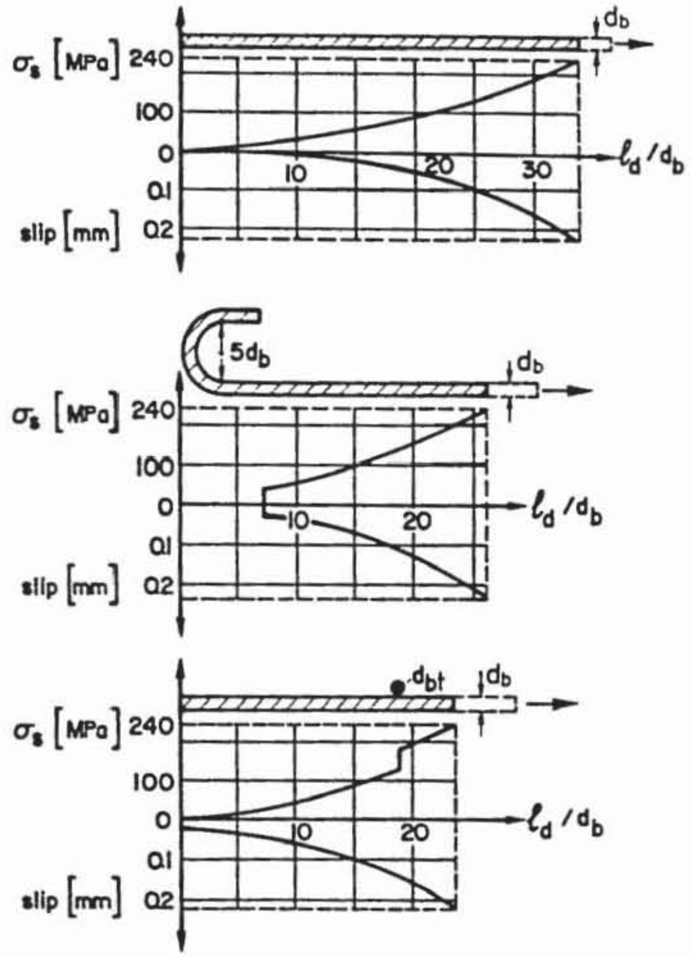

Fig. 4: Distribution of steel stress and slip along anchorage length (after /2/)

$l_{d1} = \alpha \cdot l_d$			
	Type of anchorage	α for bars in tension	α for bars in compression
1	l_{d1}	1,0	1,0
2	l_{d1} l_{d1} l_{d1}	0,7 (1,0)	1,0
3	l_{d1}	0,7	0,7
4	l_{d1} l_{d1} l_{d1}	0,5 (0,7)	1,0
5	<100 mm, l_{d1}, d_b; $d_b \leqq 16$ mm single bars; $d_b \leqq 12$ mm double bars	0,5	0,5

Fig. 5: Anchorage length according to CEB/FIP and DIN 1045

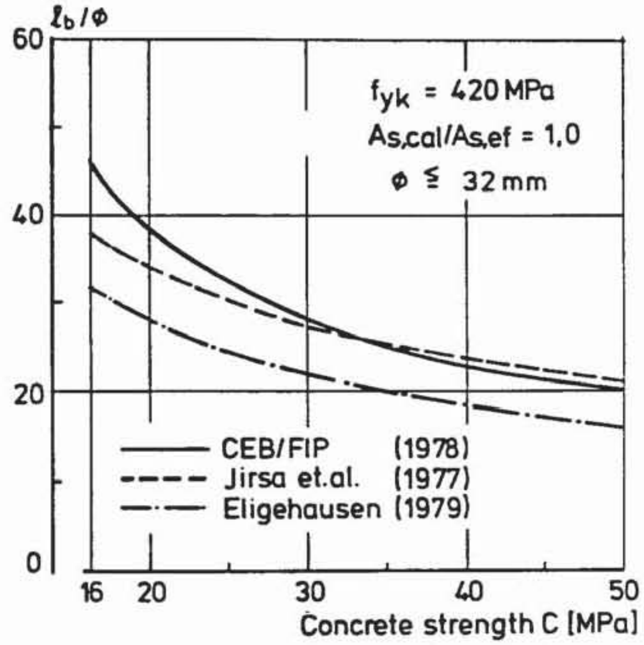

Fig. 6: Anchorage length for straight bars

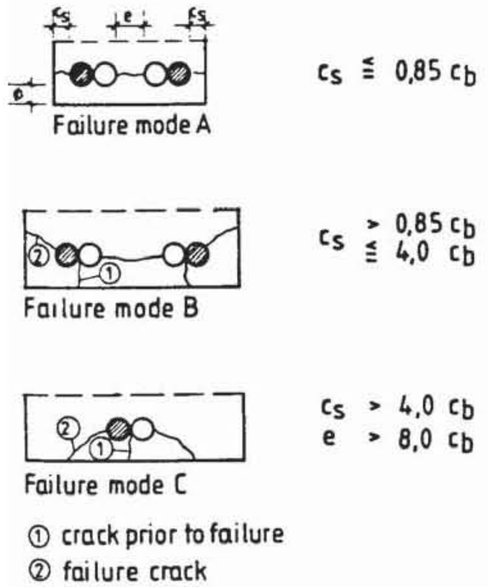

Fig. 7: Failure modes for splices in beams

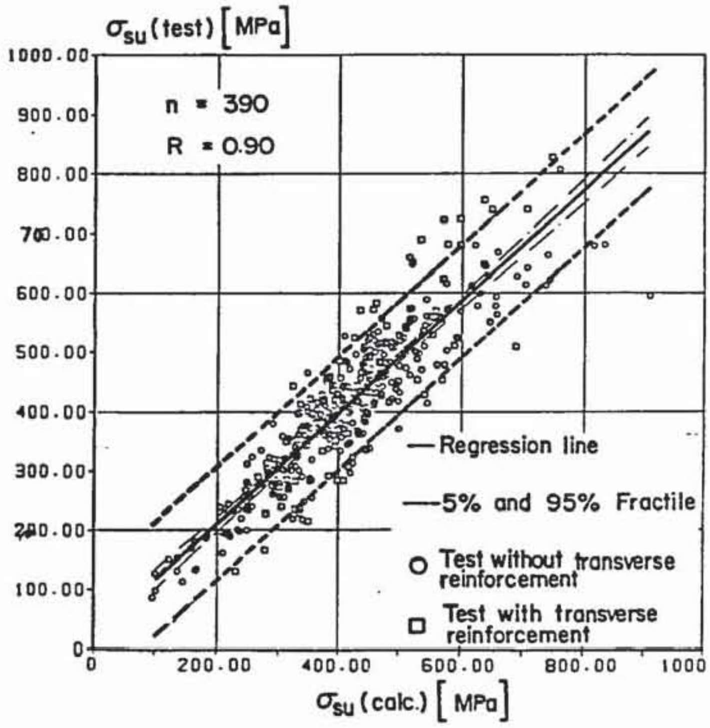

Fig. 8: Splice tests: tested vs. calculated strength (after /3/)

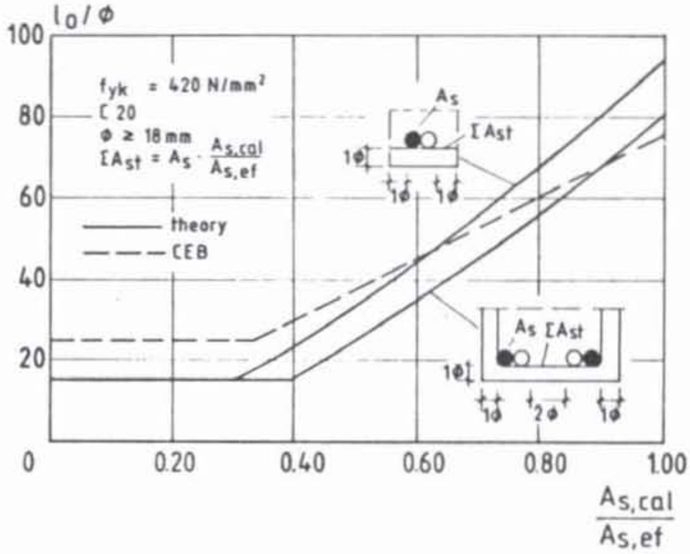

Fig. 9: Required lap length of straight deformed bars as a function of the ratio $A_{s,cal}/A_{s,ef}$

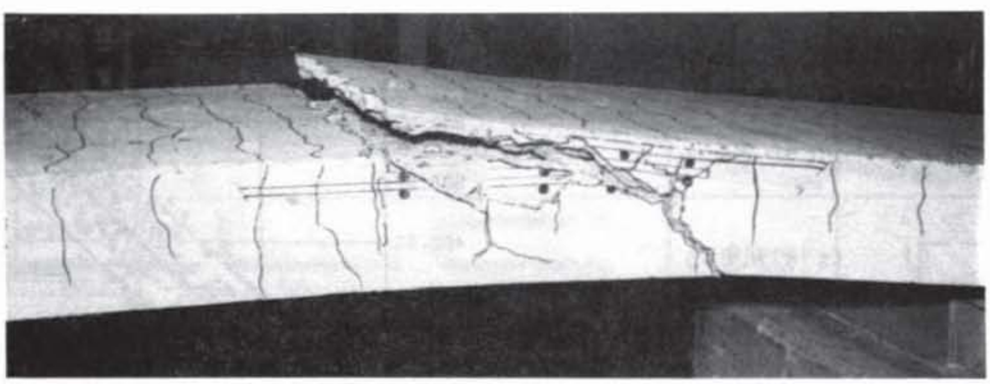

Fig. 10: Failure of a lap splice of welded wire mesh (taken from /5/)

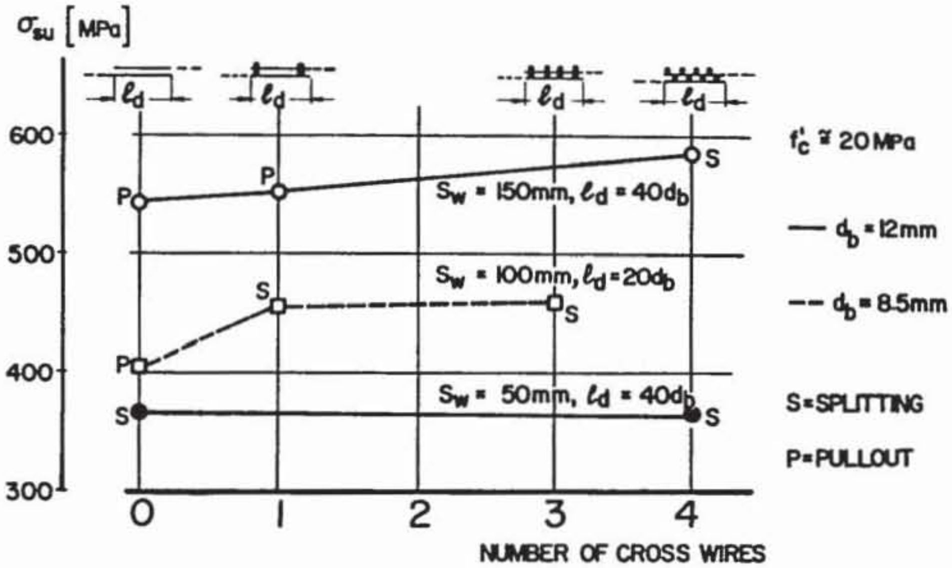

Fig. 11: Influence of cross wires on the strength of splices (after /5/)

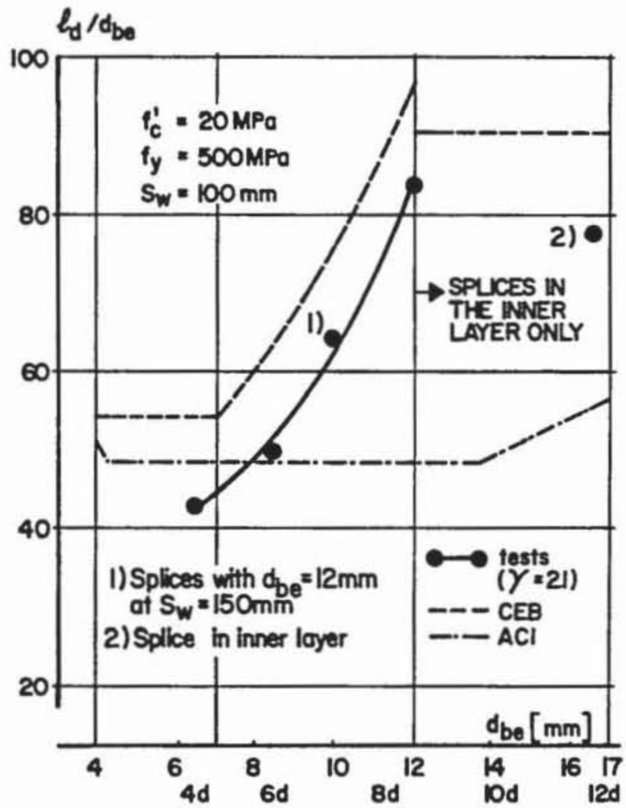

Fig. 12: Lap length for splices of wire mesh (after /5/)

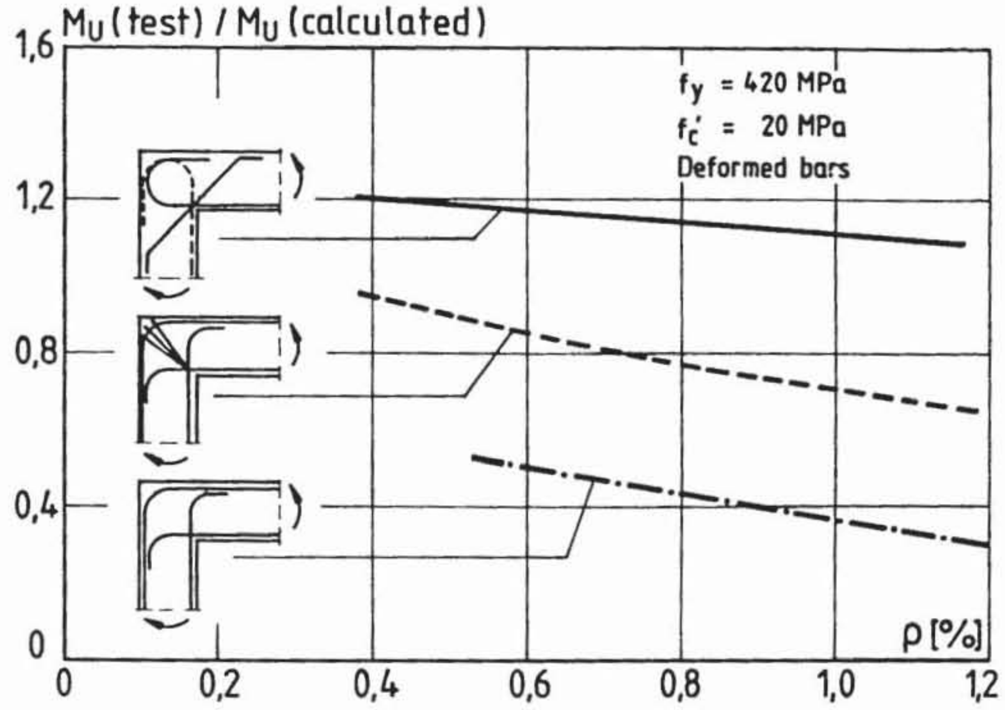

Fig. 13: Effiency of frame corners (after Kordina /6/)

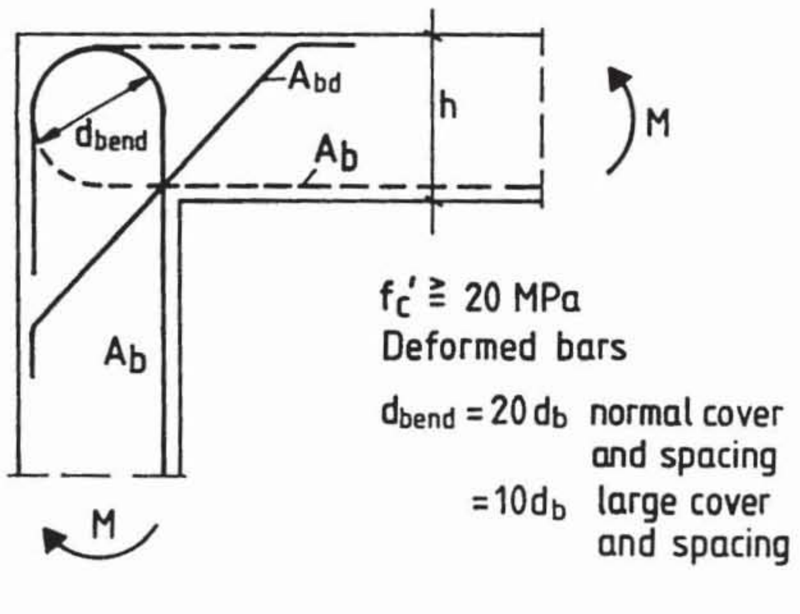

Fig. 14: Frame corner, DIN 1045 (after /7/)

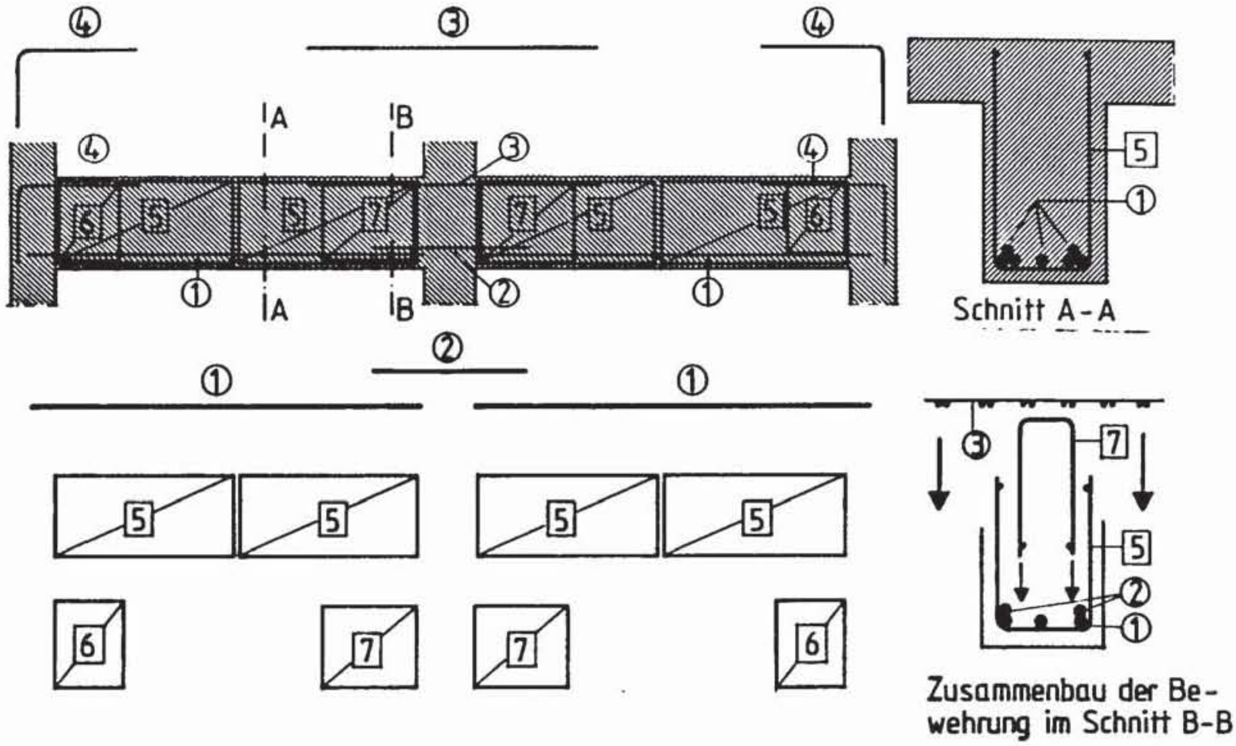

Fig. 15: Reinforcement of a continous beam using industrialized reinforcement layout, schematically (after /9/)

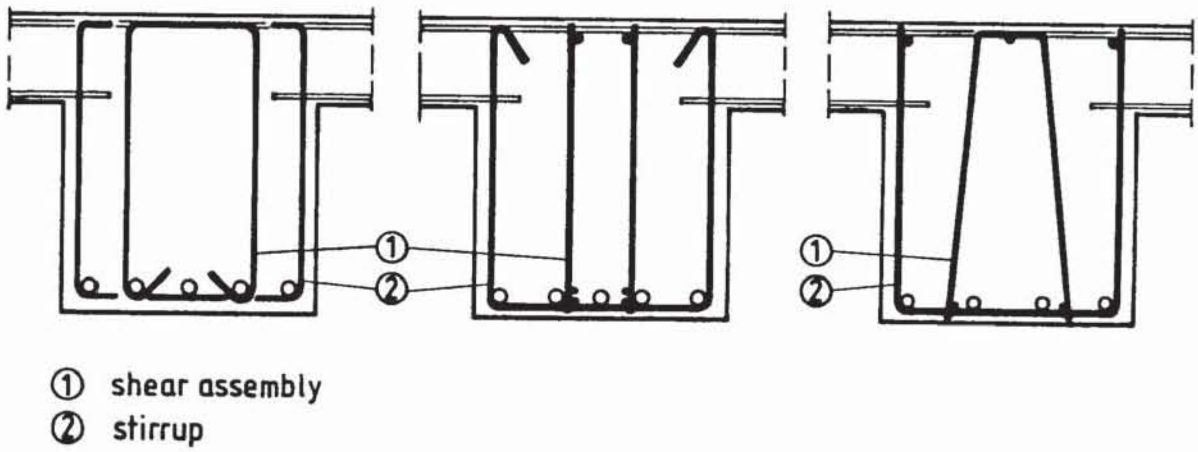

Fig. 16: Simplified shear reinforcement (after /10/)

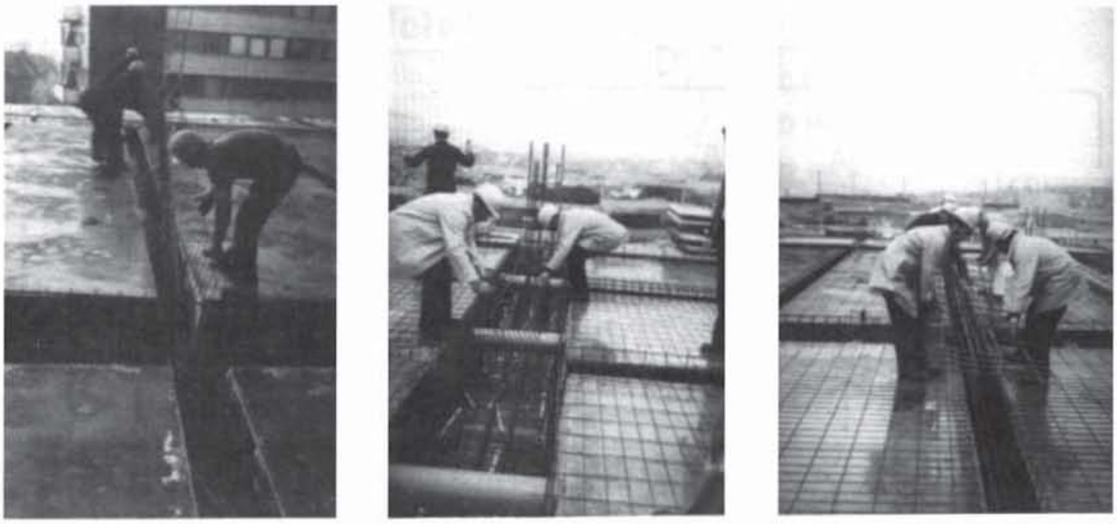

Fig. 17: Placing of the industrialized reinforcement (after /9/)

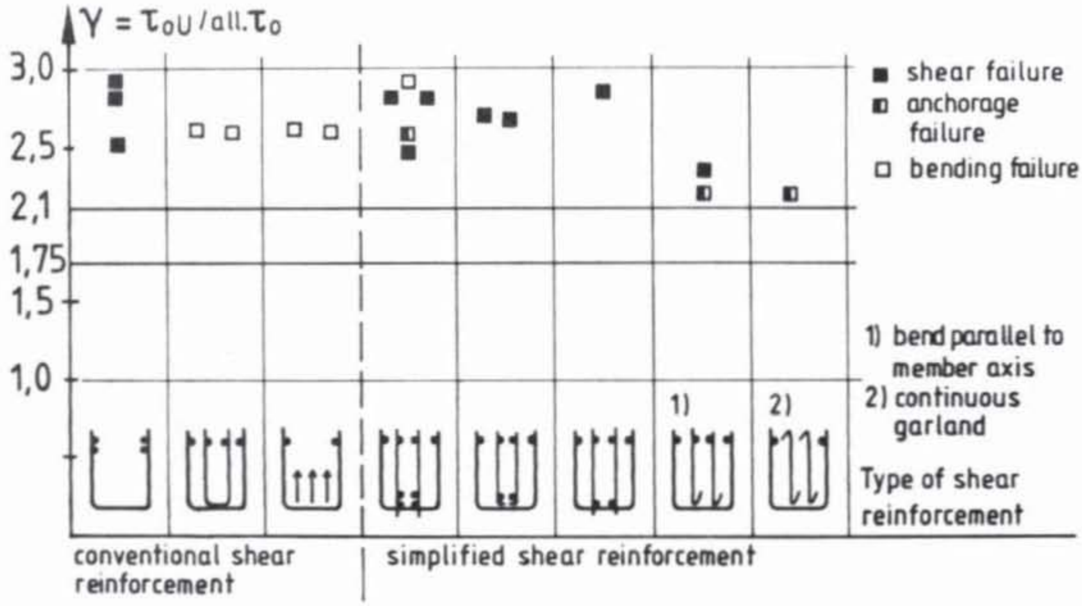

Fig. 18: Safety factors against shear failure (after /10/)