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The effect of anchorage on the effectiveness of the shear reinforcement in the punching zone

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Abstract

All building codes require well anchored shear systems in the punching zone. The enforced experimental investigation of 10 symmetric punching tests with conventional stirrups and stirrups made of fabric reinforcement shows, that the effectiveness of the shear reinforcement depends on the quality of the anchorage. This result could be confirmed by tests from the literature too. In addition, the punching design approach of DIN 1045-1, 2001 [Tragwerke aus Beton, Stahlbeton und Spannbeton, Teil1: Bemessung und Konstruktion. Beuth Verlag, Juli 2001] is presented.

Keywords: Punching shear; Shear reinforcement; Anchorage; Detailing,; Code rules

1. Introduction

Flat slabs are more and more built in buildings with a high level of technical installations or in buildings with flexible room arrangements during their life time such as offices. The main problem in practice is often to provide the resistance against a punching failure at the flat slab-column connections. The punching resistance can be increased by using a larger column diameter, a larger effective depth, more flexural reinforcement, higher concrete compressive strength or by additional shear reinforcement. Large columns are often rejected by architect. A larger slab depth increases the dead load and the cost of the footings and columns. Increase of the flexural reinforcement ratio and the concrete strength is less effective and in many cases not practical. Consequently, providing an effective shear reinforcement is the most economic solution in this situation. In the past, many shear systems were developed to prevent punching failures (double headed studs, headed stud rails, shear ladders, shear hoops,...). The economic efficiency of each shear system in the punching zone is influenced by the following criteria:

(1) reaching the material strength,
(2) anchorage behaviour within the tension and compression zone,
(3) strengthening of the compression zone at the column face,
(4) ductility,
(5) cost in terms of labour for placing simplicity and speed in assembling the flexural reinforcement and the shear system,
(6) interaction between the arrangements of shear system and the flexural reinforcement,
(7) material and production costs.

It is obviously, that the success in practice of each shear system will be increased by increasing the shear capacity and decreasing the costs. This paper presents the dependency between the punching shear capacity and anchorage behaviour of the shear system. In addition, the comparisons of eight tests with stirrups made of fabric reinforcement, two tests with conventional stirrups and with tests from the literature were presented.

2. Detailing of some kinds of shear systems

All building codes regulate that well anchored stirrups have to enclose one direction of the top and the bottom layer of the flexural reinforcement in the punching zone (Fig. 1), therefore the placing of the flexural

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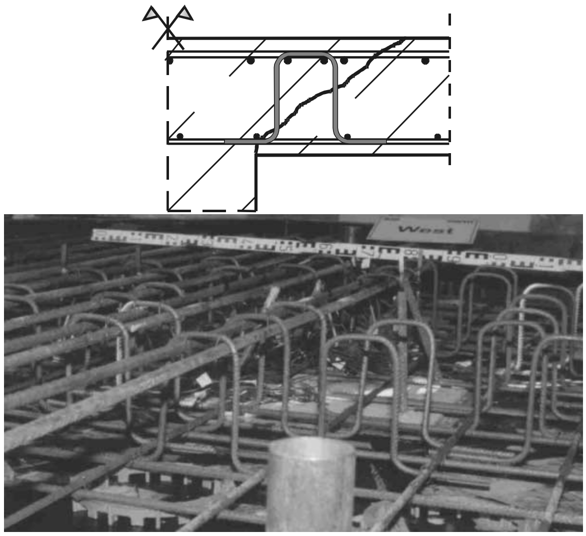

Fig. 1. Layout II – conventional stirrups with 90° bents in the compression zone.

reinforcement is costly in terms of labour on the building site.

For headed shear studs, it is sufficient that the anchors (anchor diameter = $3d_s$) are placed in the top and in the bottom layer of the flexural reinforcement, respectively. This detailing is very simple and speeds up the placing of the reinforcement on site (Fig. 2).

A comparable fast solution with stirrups is possible, only if the stirrups are partitioned into a top- and a bottom-stirrup (Figs. 3 and 4). These two parts have to be connected by a vertical splice. The placing gets particularly simple if the bottom- stirrup does not enclose the bottom reinforcement layer (Fig. 3).

In contrast to these types of detailing, shear assemblies like the Riss–Star–Elements (Fig. 5) or lattice girders (Fig. 6) were placed between the flexural reinforcement layers. Therefore, the placing of the flexural

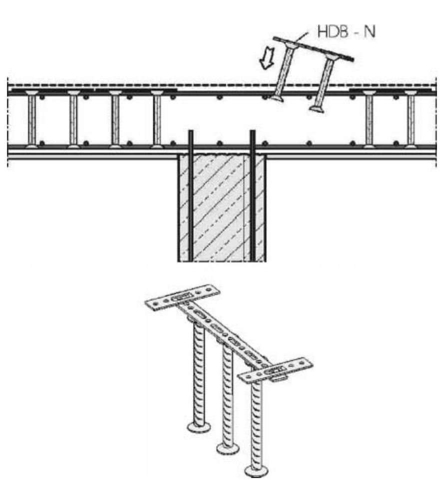

Fig. 2. Double headed studs [6].

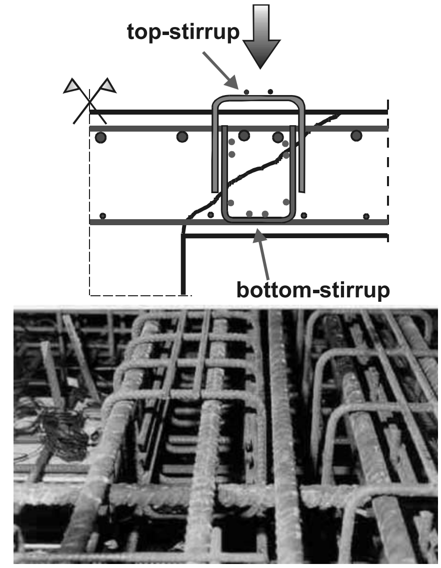

Fig. 3. Type I: stirrups made of fabric reinforcement with a vertical splice and without enclosing the bottom flexural reinforcement.

reinforcement does not depend on the arrangement of the shear reinforcement. This will be the fastest solution to place the reinforcement on site.

But, the economical success of these shear systems is also influenced by the available maximum punching shear capacity. It is well known that the maximum punching shear capacity is influenced by the items (1)–(3). In addition, it depends on the propagation of the bending cracks into the compression zone at the column face. These cracks are controlled by the flexural reinforcement. The propagation of the punching shear crack is

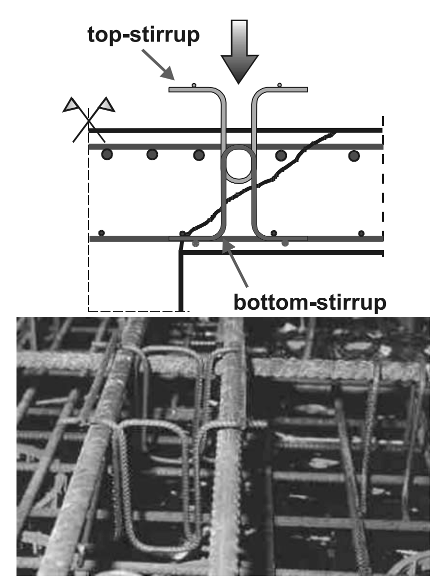

Fig. 4. Type III: stirrups made of fabric reinforcement with a vertical splice and enclosing one direction of the top and bottom flexural reinforcement.

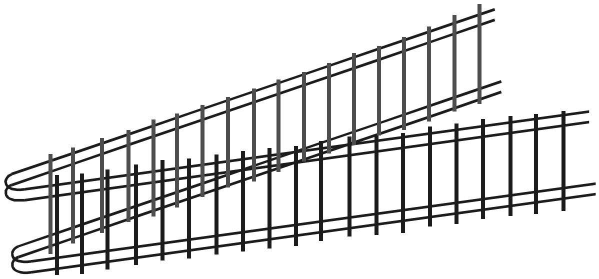

Fig. 5. Riss–Star–Ellement [8].

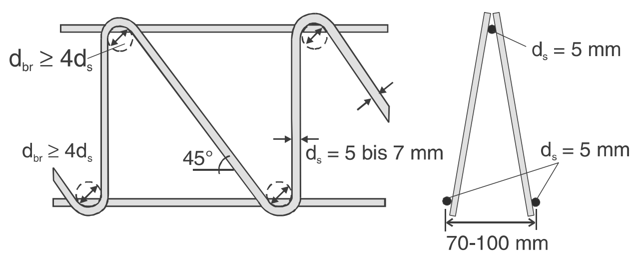

Fig. 6. Lattice girder [9].

controlled by the yield strength, bond properties and the anchorage behaviour of the shear reinforcement.

3. Anchorage behaviour of high grade and high bond steel bars

Especially in thin flat slabs it is difficult to place well anchored stirrups because:

(a) The bending moments cause large cracks in two directions and therefore, the bond strength of the high grade bars will be fundamentally reduced and the anchorage slip is increased.

(b) In the range of two times of the concrete cover, the height of the compression zone leads to a short vertical anchorage length in the compression zone.

Links can be anchored by 90°, 135° or 180° bends, transverse welded bars, headed anchors or several combinations of these elements. To classify the quality of these different anchor types, 3D-finite element simulations with the program LIMFES [4] were performed. The finite element model shows a typical pull out test from the literature. The investigated bond length corresponds to a typical crack pattern in shear reinforced punching tests (Fig. 7). For the first calculations, the investigated bond lengths were less than the mean value of the measured bond length in tension and in compression ($[166 + 64]/2 < 90$ mm, Fig. 7).

The concrete was modelled by Ottosen model [5] with 8-node brick elements. The bond behaviour of the high grade and high bond steel link was simulated by interface elements and a bond model of Eligehausen et al. [6] was used. For the link, 8-node brick elements with an elasto-plastic material model were applied.

Fig. 8 shows stress–slip relation for typical anchorage types. These relations depend on the effective bond length, the characteristic concrete strength, transverse compression or tension and the bar diameter. Therefore Fig. 8 gives only an idea of the anchorage qualities in thin flat slabs with short bond length. Starting from the assumption that an anchorage slip in the range of 0.1–0.15 mm is compatible to the rotation capacity of the compression zone at the column face [10], the following results can be observed: it was not surprising that the straight link, which is anchored only by bond, results to the lowest steel stress of approximately 250 MPa (curve A). The 180° bend without enclosing a longitudinal bar (curve C) and the 90° bend with enclosing a longitudinal bar (curve B) showed the same stress–slip relationship (maximum steal stresses = 350 MPa). The maximum link stress of 180° bends with enclosing the longitudinal reinforcement (hooks) result in 400 MPa (curve D). Transverse welded bars improved significantly the anchorage qualities. From curve E, it can be observed

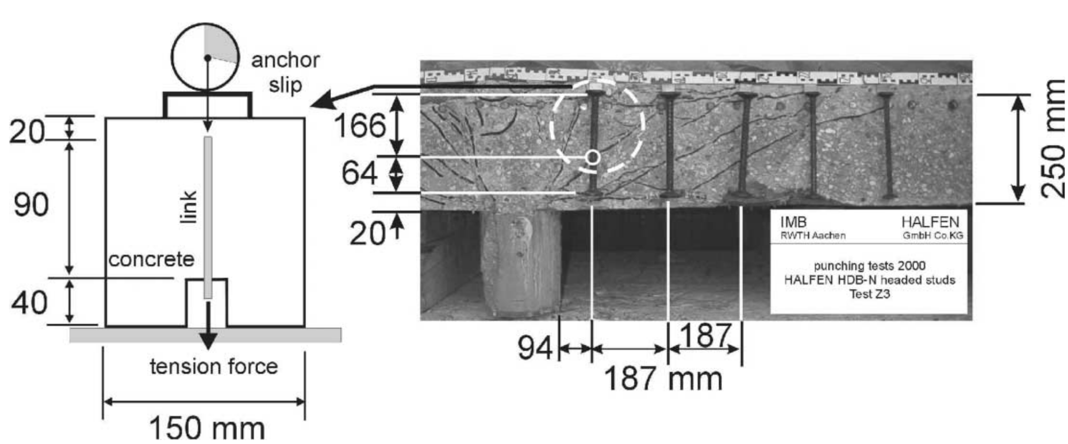

Fig. 7. Idealized pull out specimen for the shear reinforcement drawn from a saw [7] in the punching zone.

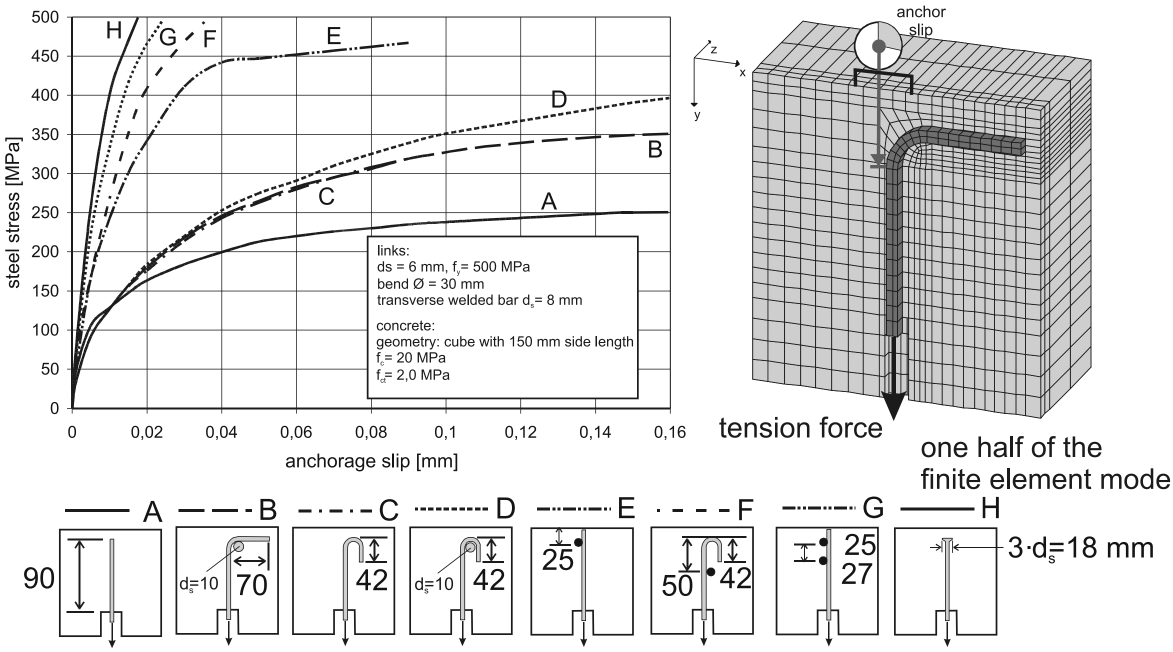

Fig. 8. Bar stress–anchorage slip-diagram for typical anchorage details of high grade, high bond steel bars.

that 90% of the yield strength could be reached at a slip value less than 0.1 mm. All combinations of bends with transverse welded bars reached the yield strength by decreasing the anchorage slip (curves F– H). From these simulations can be concluded that links with transverse welded bars are close to the high anchor quality of headed studs. The efficiency of 180° bends (hooks) and 90° bends is acceptable only if the longitudinal reinforcement is enclosed. Therefore, the detailing rules for links of e.g. Model Code 90 or Eurocode 2-Part 1 can be confirmed by finite element simulations.

Note that, it is not sufficient to define only a stiff anchor slip behaviour as the anchorage quality, because the dowel effect of the flexural reinforcement is able to induce a splitting of the concrete cover, which leads to an early combined punching and bending failure. Numerical simulation of the punching failure taking into the dowel effect of the flexural reinforcement and different anchorage types were not totally sufficient. Therefore, this phenomenon was among other investigations studied by tests.

4. Test results

The stiff anchorage slip behaviour of transverse welded bars and the higher speed for placing of the flexural reinforcement motivated a punching test program with stirrups made of fabric reinforcement (Table 2). The investigated stirrup types were named by the Roman symbols I–III. Type I is shown in Fig. 3 and type III in Fig. 4. Type Ia is equal to type I, but without any top-stirrups. Type II is a conventional stirrup as shown in Fig. 1.

The test specimens represent the full scale part of a flat plate structure extending from an interior column to the line of contra flexure for a centric loading in the column (Fig. 9(c)). The specimens had a slab thickness between 230 and 275 mm, a plate diameter of 2750 mm, a loading diameter of 2400 mm and a square column of 400, respectively, 320 mm side length (Fig. 9(a) and (b)). The mean yield strength of the reinforcement was 580 MPa. For example, the arrangement of the reinforcement of test series P2 is shown in Fig. 9. Each specimen was carefully instrumented with dial gauges and special photogrammetric points to determine deflections and rotations. One hundred electrical strain gauges were used to measure strains at selected locations on the flexural reinforcement, the stirrups and on the concrete at the column face on the bottom plate surface. The ultimate loads of the specimens and further relevant information are given in Table 1.

Each punching failure was characterised by the following points:

(a) The tension reinforcement did not reach the yield strength.

(b) The compression reinforcement was strained at a low level.

(c) The radial concrete strains were concentrated in the proximity of the column.

(d) The radial concrete strains at the column face decreased before the ultimate load was reached.

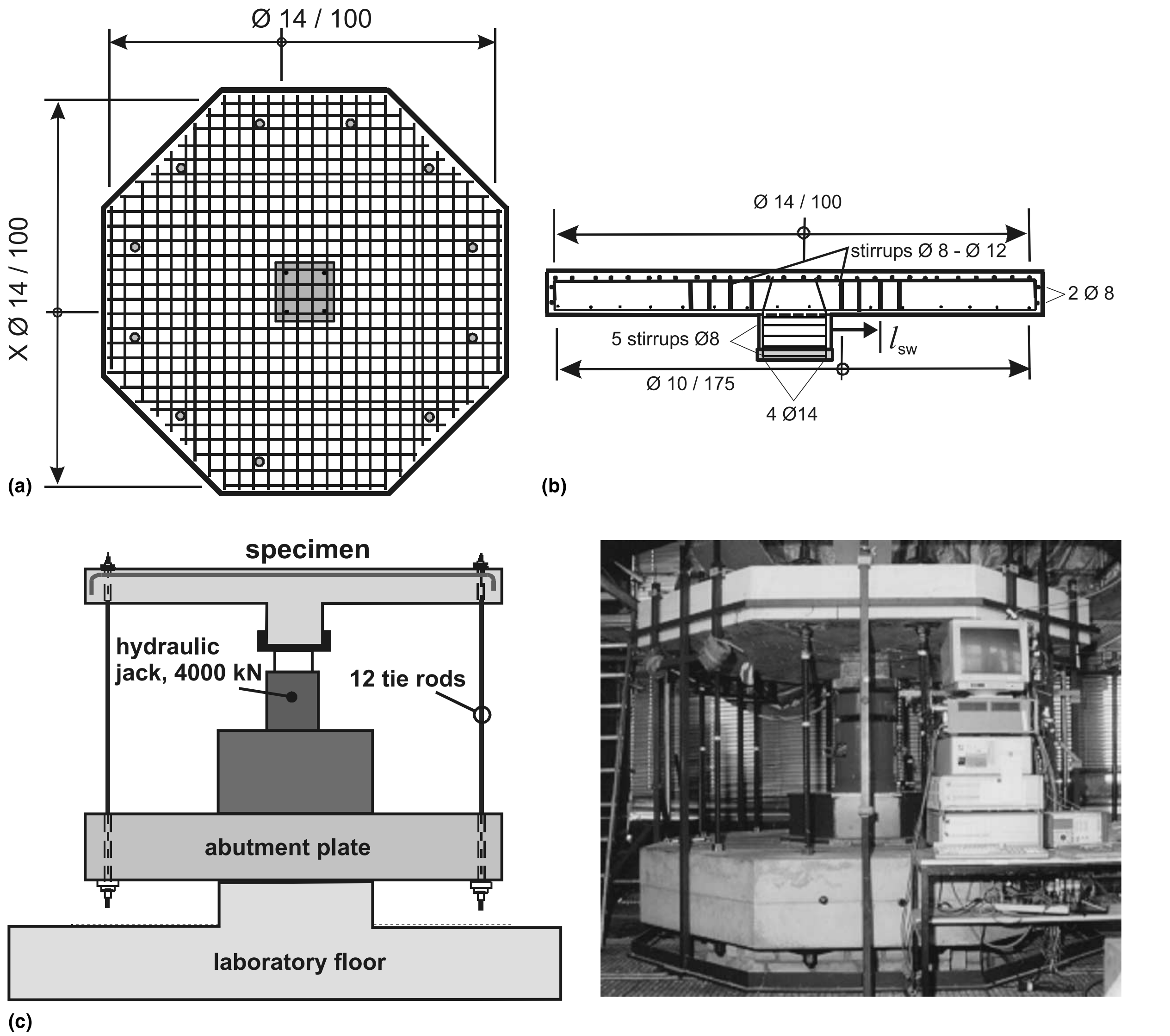

Fig. 9. Reinforcement of test series P2 and test setup: (a) top layer of flexural reinforcement; (b) cross-section of the specimens; (c) test setup.

(e) The tangential concrete strains were higher and less concentrated than the radial.

(f) The calculation of the maximum bond stresses during loading according to Model Code 90 shows that a classical bond failure was not critical.

The failure mode on the level of the maximum punching shear capacity was characterised by a concrete crushing at the column face combined with reinforcement stresses smaller than the yield strength. From the crack formation in the saw-cut of specimen PI-7 (Fig. 10), it can be seen that the concrete crushing occurs in the area of the unconfined concrete cover. Therefore, it is obvious that the maximum shear capacity is less than the maximum shear capacity of beams which fails in the area of the confined web. Due to this, some design codes (Eurocode 2-Part 1, DIN 1045,...) formulate the maximum punching shear capacity in dependency of the punching shear capacity without shear reinforcement. In addition, the shear cracks tend to be horizontal in the top layer of the flexural reinforcement. This indicated that the dowel effect of the flexural reinforcement was effective because a bond failure was not critical.

The measurements of stirrup strain and anchor slip (stirrups with 90° bends and stirrups made out of fabric reinforcement) show for all tests, that this anchorage of links in thin flat plates is not sufficient to activate the yield strength (Fig. 12). This is not contradictory to the finite element simulations of curve G in Fig. 8 because the shear reinforcement of all tests consist of diameter 10 or 12 mm with less effective bond conditions as in the simulations by using a diameter of 6 mm. In addition,

Table 1
Ultimate loads and details of the test specimens

No.[a]	D^{b} (m)	a/b^{c} (m)	$f_c{}^{d}$ (MPa)	ρ, ρ'^{e} (%)	$l_{sw}/A_{sw}{}^{f}$ (m), (cm^2)	$V_u{}^{g}$ (kN)	Failure mode
P1	0.19	0.4/0.4	21.9	0.806, 0.235	–	615	Punching
P1-Ia	0.19	0.4/0.4	27.3	0.806, 0.235	0.110/11.0; 0.20/11.0 $\sum$22.0	1151	Punching outside the shear reinforced area
P1-II	0.19	0.4/0.4	26.2	0.806, 0.235	0.100/12.57; 0.200/9.43 $\sum$22.0	1055	Punching outside the shear reinforced area
P2-I	0.19	0.4/0.4	37.9	0.806, 0.230	0.075/7.04; 0.175/7.04; 0.250/7.04; 0.350/10.05 $\sum$31.2	1326	Punching outside and inside the shear reinforced area
P2-II	0.19	0.4/0.4	29.8	0.806, 0.235	0.060/4.02; 0.172/3.07; 0.362/15.08; $\sum$32.2	1109	Punching inside the shear reinforced area
P2-III	0.19	0.4/0.4	37.5	0.806, 0.235	0.065/7.04; 0.165/7.04; 0.260/7.04; 0.365/10.05; $\sum$31.2	1276	Bending
P3-I	0.22	0.32/0.32	23.2	1.15, 0.413	0.085/6.03; 0.189/9.05; 0.355/8.04; 0.459/15.08; 0.685/5.03; $\sum$43.3	1624	Punching inside the shear reinforced area
P4-III	0.22	0.32/0.32	27.8	1.132, 0.408	0.105/6.03; 0.205/10.50; 0.365/8.04; 0.465/16.08; 0.615/4.02; $\sum$44.2	1522	Punching inside the shear reinforced area, the tests
P5-I	0.22	0.32/0.32	45.3	1.354, 0.409	0.115/15.71; 0.255/18.85; 0.400/11.00; 0.540/25.13; 0.690/18.85; $\sum$44.2	1936	were finished before the ultimate load was reached to study the internal shear crack pattern
P6-I	0.22	0.32/0.32	46.3	1.753, 0.550	0.100/22.62; 0.268/31.67; 0.428/22.62; 0.596/31.67; 0.718/22.62; $\sum$131.2	2349	Punching inside the shear reinforced area
P7-I	0.23	0.32/0.32	40.0	1.301, 0.393	0.145/18.85; 0.285/25.13; 0.455/15.71; 0.595/21.99; 0.740/15.71; $\sum$97.4	2117	Punching inside the shear reinforced area

[a] Specimen number with indication of the stirrup type, type I + III: (Figs. 5 and 6), type II: conventional stirrups (Fig. 3).
[b] Effective depth.
[c] Column side length.
[d] Concrete cylinder strength.
[e] Tension (ρ_l) and compression (ρ_l') flexural reinforcement ratio.
[f] l_{sw} is the distance between the column face and the stirrup row, A_{sw} the shear reinforcement cross-section area (each row and sum of all rows).
[g] Ultimate load.

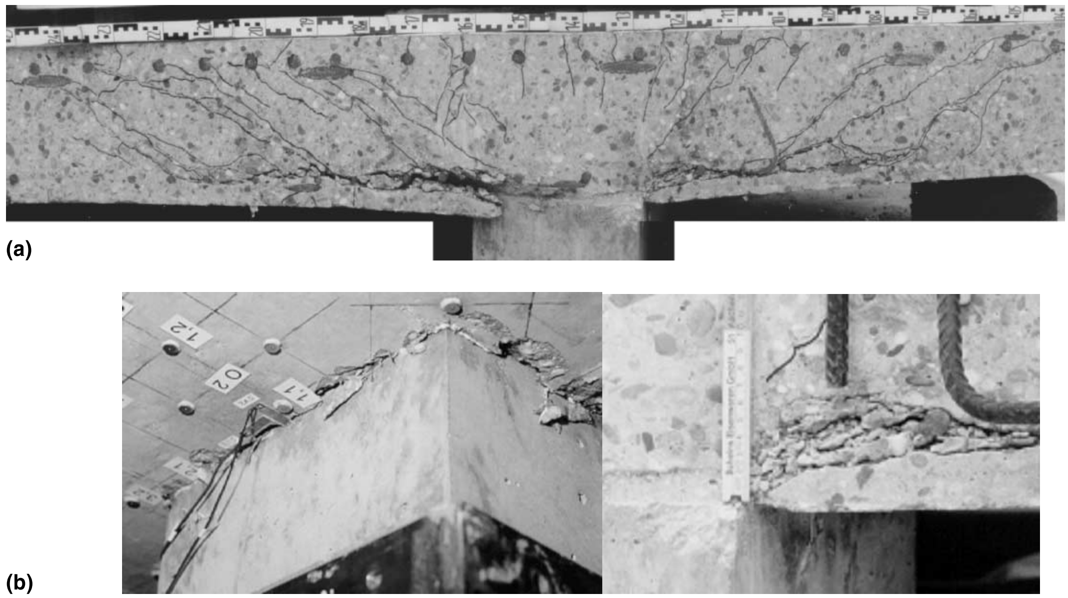

Fig. 10. P7-I: crack formation in a saw-cut after failure and concrete crushing at the column face at ultimate load: (a) saw-cut with a multiple shear crack formation in the punching zone; (b) concrete crushing of the compression zone at the column face.

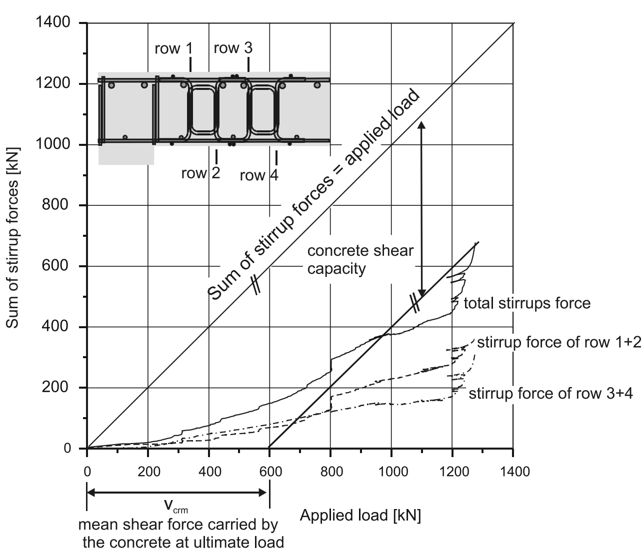

Fig. 11. P2 III: sum of stirrup forces versus applied load.

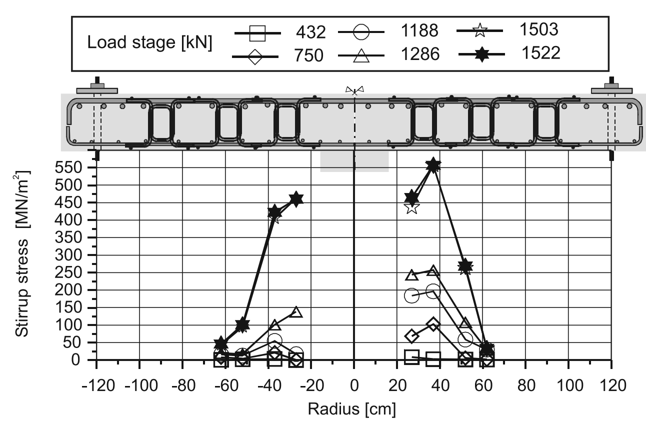

Fig. 12. P4-III: stirrup stress distribution at different load stages.

the simulations did not consider bending cracks in the area of the anchorage.

The following conclusions can be drawn from Figs. 11 and 12: (1) The stirrups in row $1 + 2$ carried the main part of the applied shear force. (2) The stirrups in row $3 + 4$ had been less activated than in row $1 + 2$. (3) The stirrups in row 3 were activated at a load stage of about 80% of the ultimate load. (4) The main function of the reinforcement in row $3 + 4$ was to avoid an early brittle punching failure outside the first stirrup rows.

A comparison of above results with tests of other shear systems has been performed by balancing all design parameters. Based on the comparison, a new empirical design approach, which has been accepted for the code revision of DIN 1045-1(2001) [1], is proposed and will be presented in the following section.

5. Design recommendation for punching DIN 1045-1

5.1. Punching shear capacity without shear reinforcement

The punching design strength without shear reinforcement is based on the principle of Model Code 90 [2] and Eurocode 2, part 1 [3]. The control perimeter is identically defined as in Eurocode 2, Part 1. Therefore, the punching shear strength is increased in comparison to Model Code 90. Special attention has to be paid to the effective flexural reinforcement ratio. It is well known from tests that the compression reinforcement does not influence the punching strength. Therefore, only the flexural reinforcement force which is balanced by the concrete strength can be considered to calculate the punching resistance (equilibrium of internal forces to balance the bending moments at the column face).

$$V_{\mathrm{Rd,ct}} = v_{\mathrm{Rd,ct}} \cdot u_{\mathrm{krit}} \ (\mathrm{MN}), \quad u_{\mathrm{krit}} = l_{\mathrm{c}} + 3 \cdot d \cdot \pi \ (\mathrm{m}), \tag{1}$$

$$v_{\mathrm{Rd,ct}} = 0.14 \cdot (100 \cdot \rho_{\mathrm{l}} \cdot f_{\mathrm{ck}})^{1/3} \cdot \kappa \cdot d \ (\mathrm{MN/m}); \quad \rho_{\mathrm{l}} \leqslant 0.40 \cdot \frac{f_{\mathrm{cd}}}{f_{\mathrm{yd}}} \leqslant 0.02, \tag{2}$$

where $V_{\mathrm{Rd,ct}}$ is the punching shear resistance; $v_{\mathrm{Rd,ct}}$ the punching shear capacity per meter; d the effective depth (m); u_{krit} the control perimeter; l_{c} the column diameter; ρ_{l} the flexural reinforcement ratio (-); κ the size effect parameter and f_{ck} is the characteristic compressive strength.

5.2. Punching with shear reinforcement

The punching shear strength is given by the minimum of: (a) the maximum punching shear capacity, (b) the shear strength within the shear reinforced area and (c) the punching shear strength outside of the shear reinforced area (Fig. 13).

5.2.1. Maximum punching shear capacity

The presented tests and tests from the literature [11] confirm that a characteristic value of the maximum punching capacity of conventional stirrups and bent-up bars is given by Eq. (3). In order to simplify the code rules, the control perimeter of the maximum shear capacity is identically defined as the control perimeter of the punching resistance without shear reinforcement. The dependence on the punching resistance without shear reinforcement has to be put down to the fact that in the first line the height of the compression zone and the compressive concrete strength are empirically considered by $V_{\mathrm{Rd,ct}}$.

$$V_{\mathrm{Rd,max}} = 1.5 \cdot V_{\mathrm{Rd,ct}}. \tag{3}$$

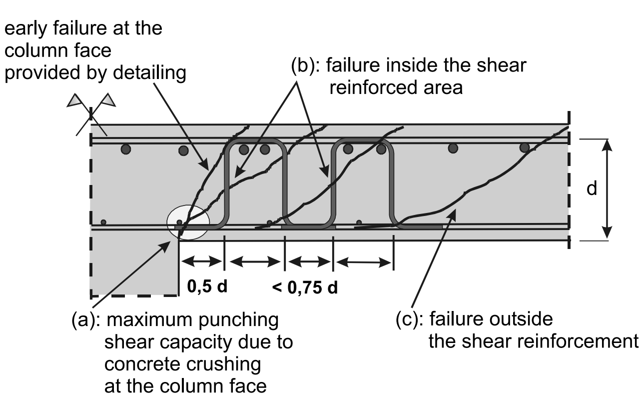

Fig. 13. Possible failure modes of shear reinforced flat slabs.

5.2.2. Punching shear strength within the shear reinforced area

The shear capacity of each stirrup row is given by a constant concrete contribution (v_{crd}) and by the shear strength of the stirrups (e.g. Fig. 11). Taking into account a stirrup anchorage slip in thin flat slabs the design strength of the shear reinforcement is appointed to 70% of the yield strength. This strength is in direct coherence to the required slab thickness of 200 mm. If the slab thickness increases the anchorage behaviour of the stirrups will be improved by longer bond lengths and leads to an increased efficiency of the stirrups (Eq. (6)).

$$V_{\mathrm{Rd,sy},i} = v_{\mathrm{Rd,sy}} \cdot u_i \ (\mathrm{MN}), \tag{4}$$

$$v_{\mathrm{Rd,sy}} = v_{\mathrm{crd}} + \frac{\kappa_{\mathrm{s}} \cdot A_{\mathrm{sw},i} \cdot f_{\mathrm{yd}}}{u_i} \ (\mathrm{MN/m}), \tag{5}$$

stirrups and bent-up bars :

$$\kappa_{\mathrm{s}} = 0.7 + 0.3\frac{d - 400}{400} \geqslant 0.7 \leqslant 1.0, \quad d \text{ in mm}, \tag{6}$$

$$\text{interior columns} : \Delta A_{\mathrm{sw}} = 2 \cdot \pi \cdot s_{\mathrm{w}} \cdot \frac{v_{\mathrm{crd}}}{\kappa_{\mathrm{s}} \cdot f_{\mathrm{yd}}}, \tag{7}$$

where $V_{\mathrm{Rd,sy},i}$ is the shear capacity in every stirrup row i; $v_{\mathrm{Rd,sy}}$ the shear capacity in every stirrup row per meter; v_{crd} the concrete contribution of the shear capacity for stirrups and bend-up bars $v_{\mathrm{crd}} = v_{\mathrm{Rd,ct}}$; $A_{\mathrm{sw},i}$ the sum of the stirrup cross-section area in each row; $f_{\mathrm{yd}} = 435$ MPa (design yield strength); κ_{s} the effectiveness factor of the shear reinforcement; u_i the perimeter of each stirrup row; s_{w} the spacing of the stirrups in the radial direction; and l_{w} is the radial distance between the column face and the last stirrup row.

The best way to explain the design procedure is to compare the acting shear force envelope line to the shear force resisting line (Fig. 14). Therefore in each stirrup row i the acting and resisting shear force has to be calculated. The design of the stirrups leads to a staggered shear force resisting line approximating the acting shear force envelope line.

5.2.3. Punching outside of the shear reinforced area

The distance between the last stirrup row and the control perimeter outside the shear reinforced area can be assumed to 1.5 d. This value is exactly equal to the distance between the column face and the control perimeter for the punching capacity without shear reinforcement. The design value of the punching resistance outside the shear reinforced area decreases from the punching resistance without shear reinforcement to

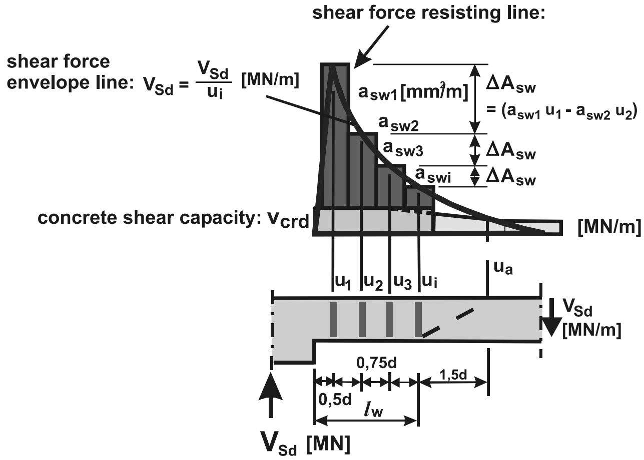

Fig. 14. Diagram of the acting and resisting shear force.

shear capacity of continuous supported slabs being 20% lower than the punching resistance (formula (10)).

$$V_{\mathrm{Rd,cta}} = v_{\mathrm{Rd,cta}} \cdot u_{\mathrm{a}} \ (\mathrm{MN}), \tag{8}$$

$$v_{\mathrm{Rd,cta}} = \kappa_{\mathrm{a}} \cdot v_{\mathrm{Rd,ct}} \ (\mathrm{MN/m}), \tag{9}$$

$$\kappa_{\mathrm{a}} = 1 - \frac{0.29 \cdot l_{\mathrm{w}}}{3.5 \cdot d} \geqslant 0.71, \tag{10}$$

where u_{a} is the exterior control perimeter (the shape of this perimeter is similar to u_{krit}).

6. Maximum shear capacity and effectiveness of different shear systems

The presented design approach considered the anchorage behaviour in two ways. First, the maximum shear capacity is scaled in comparison of the shear capacity without shear reinforcement. Second, the effective factor κ_{s} shows directly the design strength in terms of 500 MPa.

For comparisons of the shear systems it was necessary to select the tests regarding to the punching failure mode and the shear systems regarding to the anchorage type. The classification in Table 2 shows only the type of the shear system because each shear system was characterised by a typical anchorage type. Many tests [11] were performed with a yield strength of the shear system less than 500 MPa. In that case, the design strength was calculated to the minimum of the measured yield strength and the value ($\kappa_{\mathrm{s}} \cdot 500$) MPa.

From Figs. 15(a)–(f) it can be observed that the presented design approach fits well to the tests and guarantees a balanced safety level for all failure modes and shear systems. Table 2 shows the evaluated coefficients for the maximum shear capacity as a 5% value of the ratio $V_{\mathrm{Test}}/V_{\mathrm{Ru,max}}$ and the effectiveness factor κ_{s} of the anchorage. From these comparisons with tests it can be concluded: first, a punching failure outside the shear system is not influenced by the shear system (Fig. 15(f)). Second, the yield strength of high grade, high bond steel will be reached only by double headed studs, headed stud rails and hooks. Stirrups with 90° bends and shear assemblies reach only 70% of 500 MPa (Figs. 15(b) and (d)). Third, the maximum shear capacity of stirrups, bend up bars and the presented stirrup type III are on the same level. Headed stud rails enable the highest maximum punching shear capacity because of a strengthening of the concrete compression zone. The high punching capacities of all slabs reinforced with stirrup of type I (Fig. 15(e)) can be explained by three reasons: (1) the improved anchorage behaviour by welded transverse bars, (2) the double stirrup cross-section area in the shear

Table 2
Coefficient for the maximum punching shear capacity according to Eq. (3) and effective factor κ_{s} of the shear system for high grade and high bond steel with $f_{\mathrm{y}} = 500$ MPa

Kind of shear system	No. of tests with a failure outside of the shear system (-)	Maximum shear capacity		Effectiveness factor κ_{s} for high grade and high bond steel bars ($f_{\mathrm{y}} = 500$ MPa)	
		No. of tests (-)	Capacity in terms of $V_{\mathrm{Rd,ct}}$ (-)	No. of tests (-)	κ_{s} (-)
Stirrup[a]	32	25	1.50	40	0.7
bend up bar	19	18	1.50	4	~0.9
Shear assembly[b]	4	18	1.40	11	0.7
Hook[c]	6	4	~1.70	2	~1.0
Double headed bar[d]	6	17	1.65	6	1.0
Headed stud rail[d]	13	8	1.80	5	1.0
Presented stirrups					
Type I	1	4	~1.6	–	~0.7
Type III	–	2	~1.5		

[a] 90° bend with enclosing the flexural reinforcement.
[b] Transverse welded bars without enclosing the flexural reinforcement.
[c] 180° bend with enclosing the flexural reinforcement.
[d] Anchored in the layer of the flexural reinforcement.

crack because of the vertical splice, (3) partly activation of the dowel forces of the flexural reinforcement. The difference of the maximum shear capacity of stirrup type I and III showed that the stiff anchorage behaviour of transverse welded bars results in a increased shear capacity of approximately 6%. The wide scatter in the results of tests with shear assemblies (coefficient of variation 8% in comparison to 4.7% of stirrups) leads to an influence of the dowel action because by this detailing the dowel forces were only balanced by the concrete strength in tension (see Fig. 15(d): decreased linear regression by increasing the shear capacity of the assem-

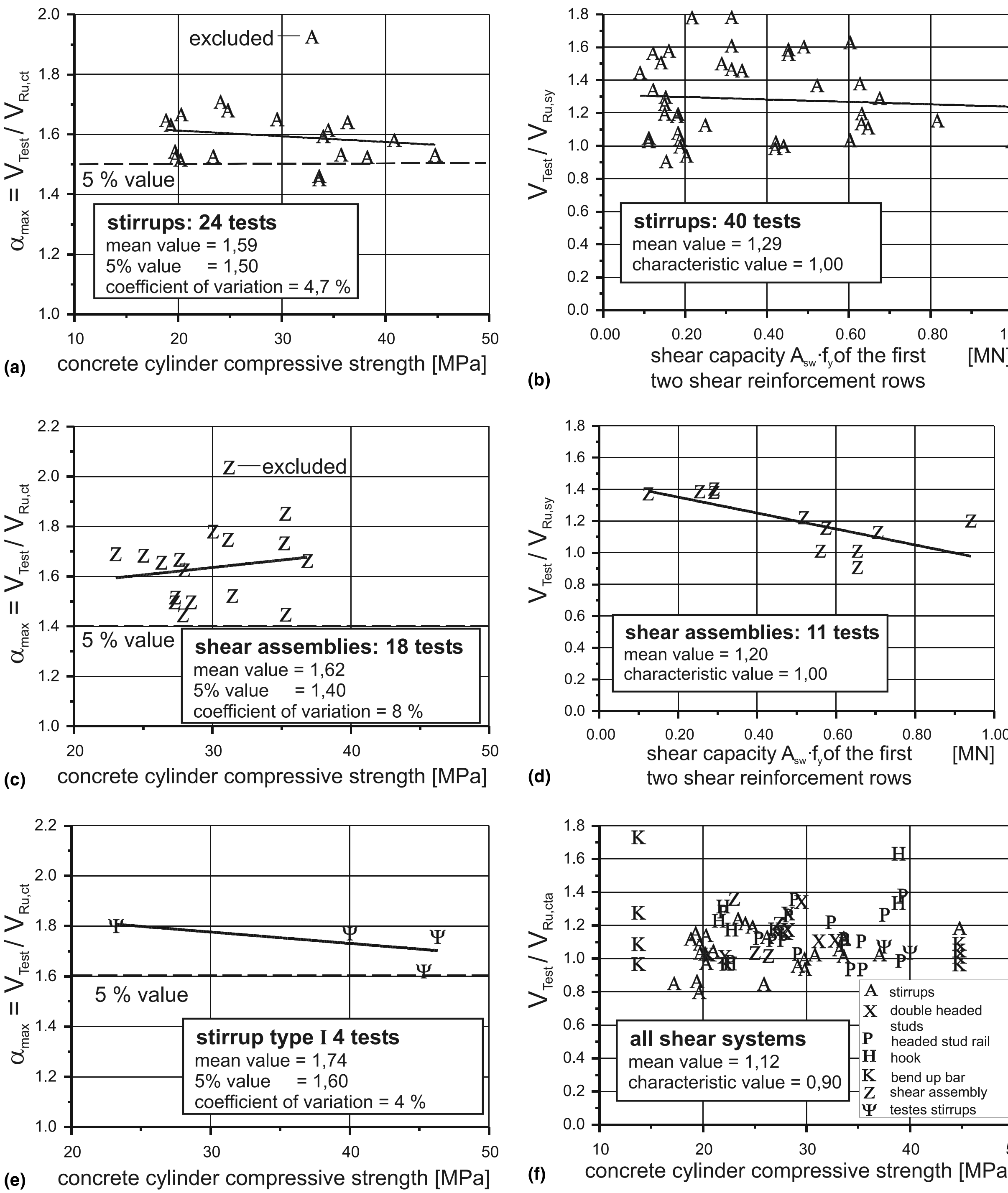

Fig. 15. Comparisons of the design approach with tests of some shear systems: (a) stirrups: maximum shear capacity; (b) stirrups: shear capacity of the shear reinforcement; (c) shear assembilies: maximum shear capacity; (d) shear assembilies: shear capacity of the shear reinforcement; (e) tested stirrups: maximum shear capacity; (f) failure outside of the shear reinforcement.

blies). Thus, the improved anchorage behaviour in combination with the simple placing of the flexural reinforcement results in an increased scatter of the failure loads. Therefore, the reduced maximum shear capacity of shear assemblies seems to be logical.

7. Conclusions

(a) The anchorage behaviour of links can be improved if transverse welded bars were placed.

(b) The results of stirrup type I showed that stirrups do not have to enclose the bottom flexural reinforcement if welded transverse bars are used as anchor elements. This conclusion has to be limited to punching tests at interior columns under symmetrical loading. Further research is needed to confirm whether this is also valid in the punching zone of edge or corner columns with moment transfer. In fact these investigations have been started in May 1999 at the Institute for Structural Concrete (IMB) at the Aachen University of Technology (RWTH Aachen, Germany).

(c) Shear assemblies speed up the placing of the flexural reinforcement in the punching zone but the maximum shear capacity is 7% less than the maximum shear capacity of conventional stirrup.

(d) The low differences in the shear capacity of stirrups and shear assemblies with transverse welded bars lead to the improved anchorage behaviour and the dowel effect of the flexural reinforcement balanced by the concrete strength in tension.

(e) In comparison to conventional stirrups, stirrups made out of fabric reinforcement, which were divided into a bottom and top stirrup, increase the maximum punching shear capacity in the range of 7%. In addition, these stirrups speed up the placing of the flexural reinforcement in the same range as shear assemblies, but the placed shear reinforcement weight will be doubled.

(f) The presented empirical punching design approach of the new German design code DIN 1045-1 (2001) is adaptable to various shear systems because all punching failure modes and the anchorage behaviour of the shear reinforcement are considered.

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