Highly Effective Lattice Punching Shear Reinforcement

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Synopsis: Lattice girders consist of longitudinal reinforcing bars which are connected to vertical or inclined struts by welding. Due to the great stiffness of the anchorage of the struts, this kind of reinforcement works well as shear reinforcement also in two way span flat slabs. The experience with different kinds of girders as punching shear reinforcement led to an optimized girder shape. This highly effective girder was tested in full scale tests to obtain a European Technical Approval based on the European design code Eurocode 2. These tests are described in this paper. The results are evaluated according to the American design rules of ACI 318-14 too. The punching shear resistance of slabs with this special lattice punching shear reinforcement is compared with the resistance when using other reinforcement.

Keywords: flat slab, lattice girder, punching shear, semi precast slab, shear reinforcement, slab-column connection

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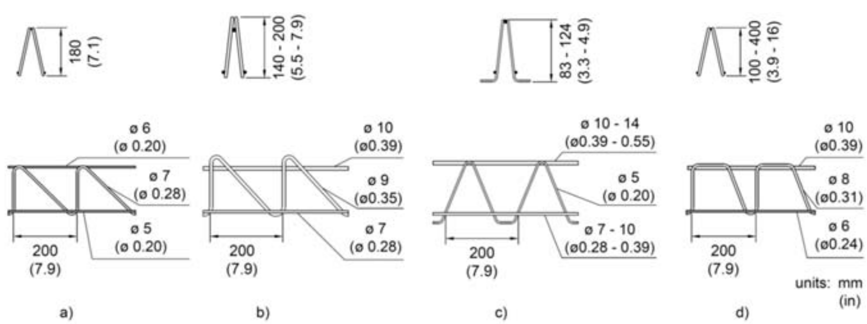
LATTICE GIRDERS AS PUNCHING SHEAR REINFORCEMENT

Lattice girders are well known in Europe and some other countries as shear reinforcement in composite slabs made of thin precast slabs and in situ topping. The struts of the lattice girders act as shear reinforcement in the final stage of the concrete slab. These slabs are also used in two way span slabs supported by columns only. In this application, the lattice girders near the column act as punching shear reinforcement. First full scale tests (Furche 1997) were carried out with a type of lattice girder (Fig. 1a) 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 ascending lattice struts to the column. The upper flexural reinforcement was put 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. This increase of the shear resistance by using lattice girders was the same as for stirrups according to the German standard at that time.

Eligehausen et al. (2003) tested lattice girders with protruding loops over the upper chord (Fig. 1b). These loops protruded into the layer of the upper flexural reinforcement. The punching shear resistance was increased by these girders with single factors of 1.79 to 1.83 (Furche 2007). The approved figure according to the German design standard valid at that time was 1.7. The increasing factor for stirrups as punching shear reinforcement given in the regarded German design standard was 1.5.

Park et al. (2007) tested lattice girders which are commonly used in composite slabs with metal sheets and in situ concrete topping (Fig 1c). These lattice girders with alternating inclined and declined struts were installed parallel to each other. The upper reinforcing layer was put on top of the upper chord. It is remarkable that the loops protruded over the lower chord into the concrete cover down to the lower surface of the slab. The punching shear resistance in the full scale tests was 1.31 to 1.50 times higher than that of the tested slabs without lattice girders.

Häusler (2009) tested slabs with lattice girders shown in Figure 1d. These girders were developed for the application as punching shear reinforcement. The punching shear resistance was increased by factors between 1.70 and 2.34 with an average value of 1.84 according to the German design rules of 2008.



a) Furche (1997) b) Eligehausen et al. (2003) c) Park et al. (2007) d) Häusler (2009)

Figure 1 — Different types of lattice girders formerly tested as punching shear reinforcement

FILIGRAN PUNCHING SHEAR REINFORCEMENT

Previous investigations indicate that lattice girders can be used to increase the punching shear resistance of two way span slabs. Beutel and Hegger (2002) showed that the stiffness of the anchorage of the transverse shear reinforcement influences their effectiveness. Stiff anchorage ensures a higher utilization of the reinforcement up to the yield strength. And the stiff anchorage ensures small cracks in the slab near the punching zone which activates greater grain interlocking. Both effects increase the punching shear resistance of the slab.

Ribbed reinforcement with welded transverse bars shows a stiff anchorage. Figure 2 shows different load-slip curves for several anchorages. Ribbed bars with a welded transverse bar and a bending loop behind the transverse bar show according to Furche (2007) a stiffer anchorage than a headed stud with a head diameter three times the bar. Therefore lattice girders with such described anchorage are suitable for application as punching shear reinforcement.

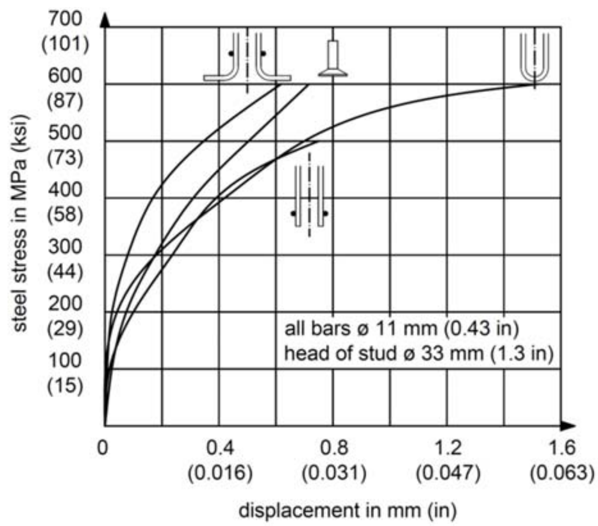


Figure 2 — Steel stress slip curves for different anchorages according to Furche (2007)

The investigations with different types of lattice girders showed furthermore that their geometry affects the punching shear capacity. An optimized reinforcement should protrude into the flexural reinforcement layer as far as possible. The inclination of the struts should penetrate the predicted shear cracks. Reinforcement approximately perpendicular to the inclined cracks is assumed to be most effective.

Based on this knowledge, a new type of lattice girder as punching shear reinforcement was developed. This type (Fig. 3) is named here Filigran Punching Shear Reinforcement. This reinforcement has two longitudinal lower chords and one upper chord with diameter of 10 mm (0.39 in). The strut diameter is 9 mm (0.35 in). The characteristic yield strength of the steel is 500 MPa (72.5 ksi). The characteristic figure (5% quantile) for the ratio of the tensile strength over yield strength is 1.05 and for the elongation at maximum strength 2.5%. All bars are ribbed.

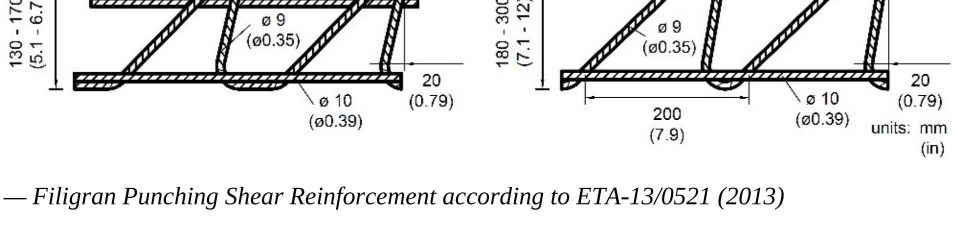


Figure 3 — Filigran Punching Shear Reinforcement according to ETA-13/0521 (2013)

FULL SCALE PUNCHING SHEAR TESTS

Full scale punching shear tests with this shear reinforcement according to Fig. 3 were conducted at the University of Aachen, Germany. In these tests the concrete strength, the thickness of the slab, the dimension of the column, the ratio, and the dimension of the flexural reinforcement were varied. Five tests on inner column slab connections were designed to estimate the maximum punching shear resistance in case of concrete failure. In these tests, high strength reinforcement with yield strength f_{yt} = 900 MPa (130 ksi) for ϕ15.5mm (#5/8) and f_{yt} = 950 MPa (138 ksi) for ϕ26.5mm (#7/8) was chosen for the upper flexural reinforcement to avoid flexural failure. Only in this manner is it possible to determine the maximum strength of the compressive concrete strut, which characterizes the maximum shear resistance in several design models. Figure 4 shows the test arrangement. This arrangement was used in tests with other punching shear reinforcement such as headed studs, too (Kueres et. al 2016).

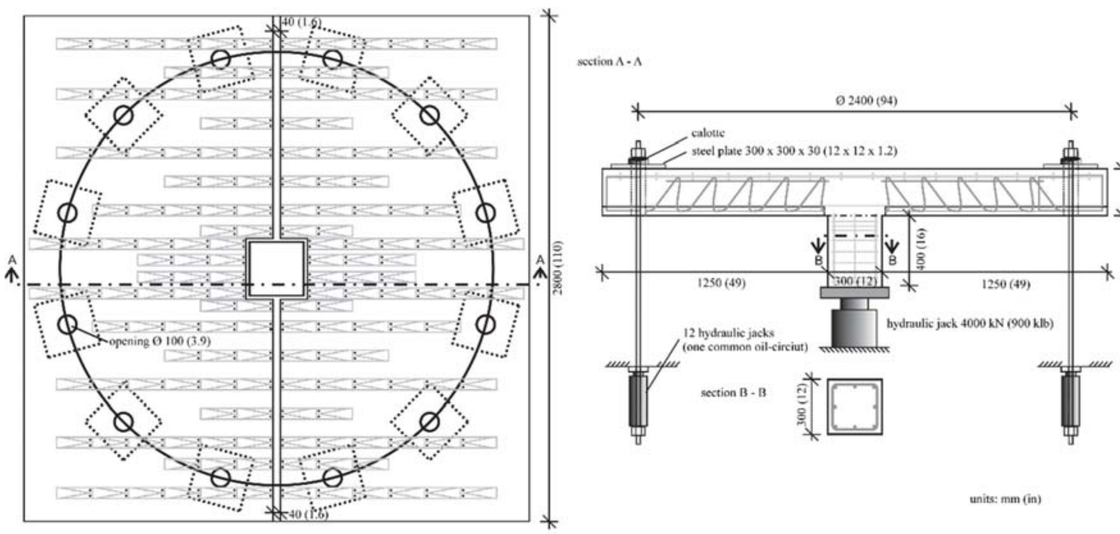


Figure 4 — Test arrangement

The single elements of shear reinforcement were positioned parallel to each other. This arrangement meets the requirement of easy installation into precast plates reinforced with parallel aligned assembly lattice girders. The test specimen was produced as composite slabs in two horizontal layers. The thin precast lower parts of the slabs were poured in the test laboratory. After hardening of these parts the upper concrete layer was poured on top. All concrete was mixed of gravel with a maximum size of 16 mm (0.6 in.). To test such a composite slab is on the safe side in comparison with a solid slab. At first two (see Fig. 5 left) or four plates respectively for each member were concreted with a thickness of 50 mm (2 in.). The surface of these plates was not mechanically treated and therefore classified as "smooth" according to Eurocode 2 (2010 and 2013). The lower parts of the slab contained the lower reinforcement and the punching shear reinforcement. These thin prefabricated plates were arranged with 40 mm (1.6 in.) wide gaps between each other. The distance between these precast plates to a prefabricated part of a column varied between 20 mm (0.79 in.) and zero. In one case the precast plates were placed 10 mm (0.39 in.) onto the column.

The gaps between adjacent prefabricated slabs were filled when the upper concrete layer was poured. Load transfer of compression forces over the joint is ensured by filling these gaps. Due to this mode of production, the test specimen contained a bond surface between the two concrete layers and joints between adjacent precast plates like a semi precast slab used in practice.

Figure 5 and Figure 6 show an example of a test specimen with the tested punching shear reinforcement.

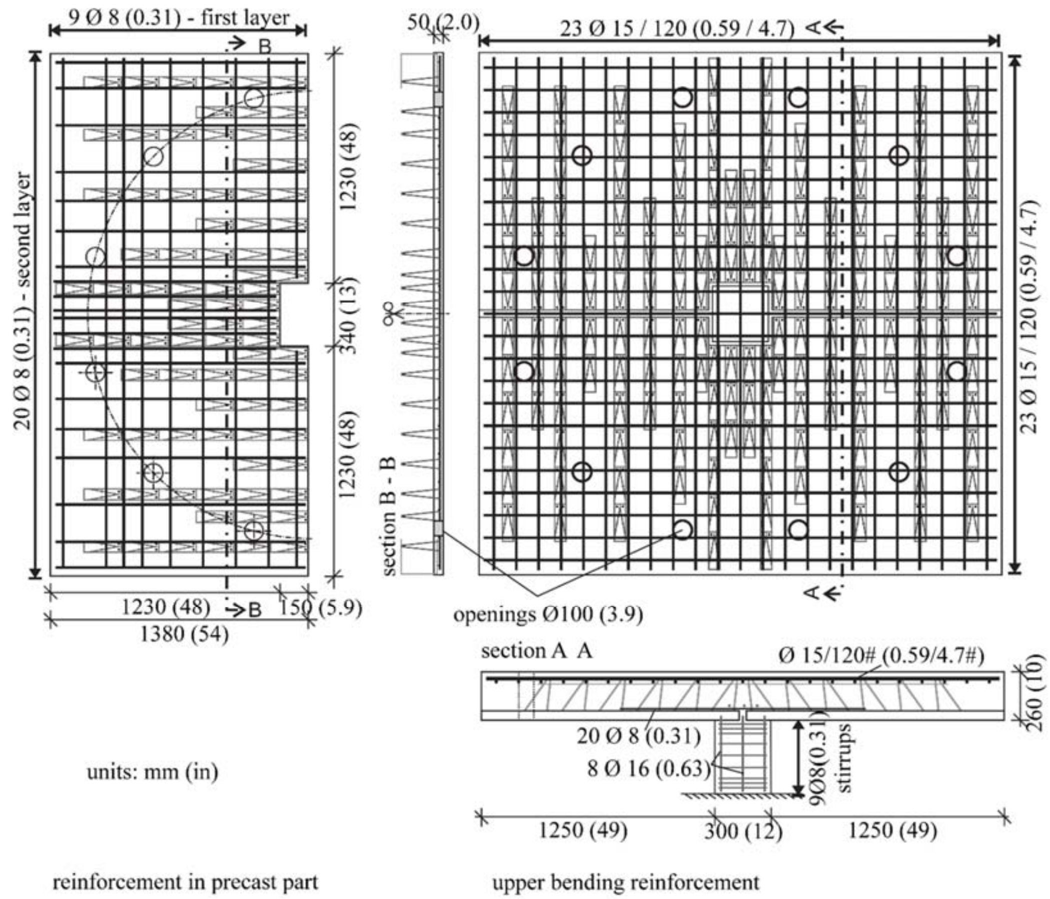


Figure 5 — Test specimen with Filigran Punching Shear Reinforcement (example FDB II 21/3)

Figure 6 — Test specimen no. 3 with FDB II 21/3 before concreting the top layer

Table 1 provides a summary of the parameters and the results of the tests. The slab thickness varied between h = 180 mm (7.1 in.) and 360 mm (14 in.) belonging to effective depths between d = 145 mm (5.7 in.) and 295 mm (12 in.). The column with a square cross-section had widths ranging from c = 240 mm (9 in.) to 300 mm (12 in.). This results in ratios of column perimeter to effective depth of $u_0/d = 4.0$ to 8.0. The ratios of the flexural reinforcement were chosen between $\rho_l = 0.70\%$ and 1.47%.

The concrete compressive strengths listed in Table 1 show the average cylinder strength f_{cm} of the upper concrete layer at the time of testing. The strength of the corresponding prefabricated lower layer varied slightly from that of the top layer, with a maximum deviation of ± 7.7 MPa (1.1 ksi).

Table 1 — Test parameters and results with Filigran Punching Shear Reinforcement

Test	Type [1)]	h	d	c	u_0/d	ratio ρ_l	f_{cm}	s_{max}	V_{test}	EC2 / ETA V_{cal}[2)]	EC2 / ETA V_{cal}/V_u	ACI 318-14 V_{cal}[3)]	ACI 318-14 V_{cal}/V_u
		mm (in)	mm (in)	mm (in)		%	MPa (ksi)	mm (in)	kN (klb)	kN (klb)		kN (klb)	
1	FDB II 14/3	180 (7.1)	145 (5.7)	240 (9.4)	6.6	0.97	26.0 (3.8)	63.2 (2.5)	896 (201)	403 (91)	2.22	380 (85)	2.36
2	FDB II 14/3	180 (7.1)	150 (5.9)	300 (12)	8.0	1.47	44.5 (6.5)	29.4 (1.2)	1461 (328)	650 (146)	2.25	600 (135)	2.43
3	FDB II 21/3	260 (10)	209 (8.2)	300 (12)	5.7	0.70	21.9 (3.2)	32.7 (1.3)	1428 (321)	661 (149)	2.16	664 (149)	2.15
4	FDB II 30/5	360 (14)	295 (12)	295 (12)	4.0	0.80	22.3 (3.2)	25.7 (1.0)	2796 (629)	1158 (260)	2.42	1096 (246)	2.55
5	FDB II 30/6	360 (14)	295 (12)	300 (12)	4.1	1.07	48.2 (7.0)	42.6 (1.7)	4121 (926)	1718 (386)	2.40	1625 (365)	2.54
										Average:	2.29		2.41
										Coefficient of variation:	0.049		0.068
										5% quantile:	2.09		2.12

1) overall height in centimeters / protrusion of upper loop in centimeters
2) expected shear strength according to eq. (1) with $f_{ck} = f_{cm}$ - 4 MPa
3) expected shear strength according to eq. (3) with $f_c` = f_{cm}$

Table 1 shows the measured maximum punching shear load V_u and the corresponding deflection of the slab. The deflection s_{max} was measured on a circle with a diameter of 2400 mm (94 in.) around the column. The loads of 12 linked hydraulic cylinders were also applied on this circle.

Figure 7 shows the deflection curves of the tests. The deflection at maximum load between 25 mm (1 in.) and 63 mm (2.5 in.) gives a good impression of the high ductility of the column slab connection. Figure 8 shows the crack pattern of a slab after testing. The first cracks occurred when the load reached about 11% to 20% of the ultimate load. The first cracks ran radially to the column. Later the cracks also ran tangentially around the column. Up to 2/3 of the maximum load the load defection behavior was quite linear. Plastic deformation was observed on the level of the maximum load. After load relief a reload was possible up to 90% of the maximum load. Therefore the load bearing behavior can be described as ductile.

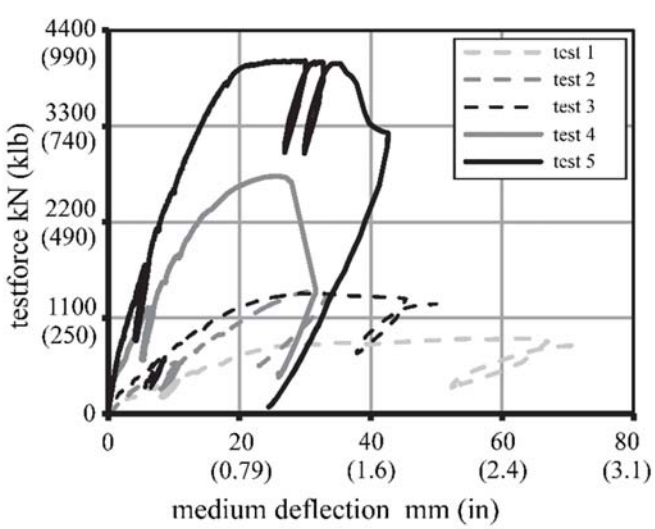


Figure 7 — Load deflection curves of the slabs

Figure 8 — Crack pattern of the upper surface of test no. 3 in Table 1

After the tests had been carried out, the specimen was cut to observe the crack pattern inside the slab (Fig. 9). The composite joint between the lower and the upper concrete layer shows no initial crack. The compressive zone was split due to the high compressive stress. The horizontal crack occurred below the bond joint on the level of the lower reinforcement layer. The shear cracks were finely distributed, small and penetrated by the shear reinforcement. A few load bearing bars of the shear reinforcement adjacent to the column yielded with a strain above 0.25%. The shear cracks proceeded into the layer of the upper flexural reinforcement. The failure mode was a punching shear failure. The strain of the flexural reinforcement did not reach the yield strain.

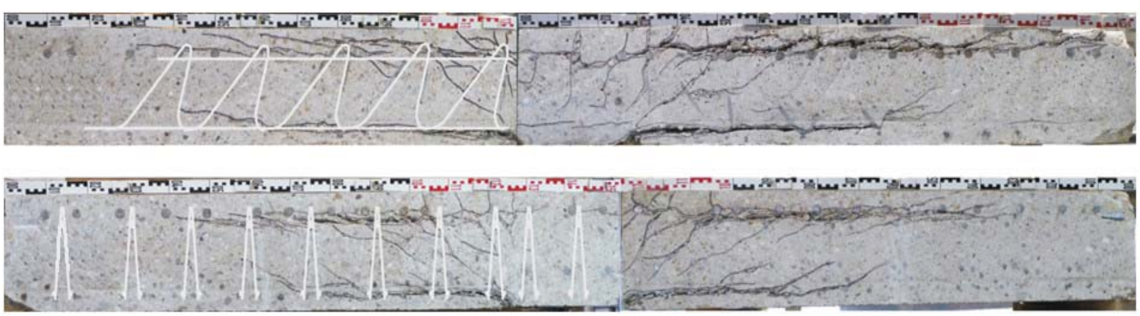
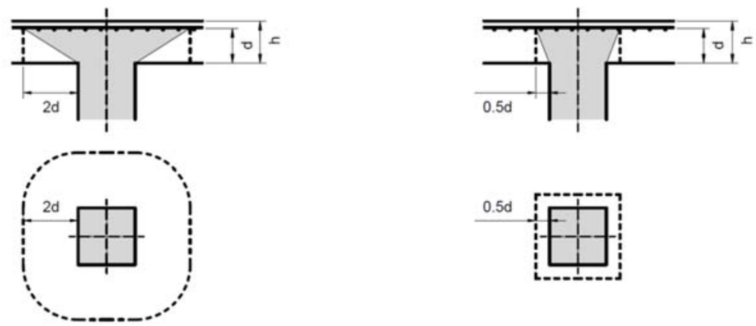


Figure 9 — Saw cut of a test specimen after testing

EVALUATION ACCORDING TO EUROCODE 2

According to the design concept of DIN EN 1992-1-1: Eurocode 2, the maximum punching shear strength must be proved in the perimeter of the column. According to national adaptations of this standard like DIN EN 1992-1-1/NA, a deviated control perimeter around the column at a distance of 2.0d is used (Fig. 10a). This design concept applies in European Technical Approvals to special punching shear reinforcement such as double headed studs (e.g. ETA-12/0454) or lattice girder (ETA-13/0521) too.



a) according to DIN EN 1992-1-1/NA b) according to ACI 318-14

Figure 10 — Different control perimeter for proving the punching shear resistance

For this circle the design resistance of concrete slabs without shear reinforcement is given in DIN EN 1992-1-1/NA. Equation (1) is used for the evaluation of the tests. The shear resistance depends on a factor k for the size effect, the ratio ρ_l of flexural reinforcement and the characteristic compressive concrete strength f_{ck}. This equation is valid for inner columns, slabs without normal stress, column perimeter greater than 4d and smaller than 12d and without taking into account the given minimum punching shear strength. For other boundary conditions, please refer to the mentioned standard or the ETA-13/0521.

$$v_{Rd,C} = C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} \quad \text{(MPa)} \qquad (1)$$

$v_{Rd,C}$ = design punching shear resistance

$C_{Rd,c} = 0.18 / \gamma_c$ (partial safety factor for concrete $\gamma_c = 1,5$)
$k = 1 + \sqrt{(200/d)} \leq 2$ (d in mm)
ρ_l = ratio of longitudinal reinforcement
$\rho_l = \sqrt{(\rho_{lz} \cdot \rho_{ly})} \leq 0.02$
$\leq 0.5\ f_{cd}/f_{yd}$
f_{ck} = characteristic concrete strength

$$v_{Rd,max} = \alpha \cdot v_{Rd,C} \qquad (2)$$

The maximum punching shear resistance for slabs with punching shear reinforcement is given by equation (2). The increasing factor α in this equation depends on the punching shear reinforcing system. For stirrups a factor of $\alpha = 1.4$ is given in the DIN EN 1992-1-1/NA. The figure for other reinforcement has to be proved by tests. Based on this, an increasing factor of $\alpha = 1.96$ was approved (ETA-12/0454) for double headed studs. Table 1 shows the ratio of the tested resistance to the expected resistance according to eq. (1), taking into account a characteristic concrete strength of $f_{ck} = f_{cm} - 4$ MPa with f_{cm} taken as the average strength of the tested cylinders. The ratios give single values between $\alpha_i = 2.16$ and $\alpha_i = 2.42$. With the average value of 2.29 and a coefficient of variation 0.049, a characteristic value of $\alpha = 2.09$ can be evaluated for the 5% quantile with a probability of 75%. This factor is approved in ETA-13/0521.

Figure 11 shows the results depending on the tested flexural reinforcement ratio in comparison with the approved values for other reinforcement systems.

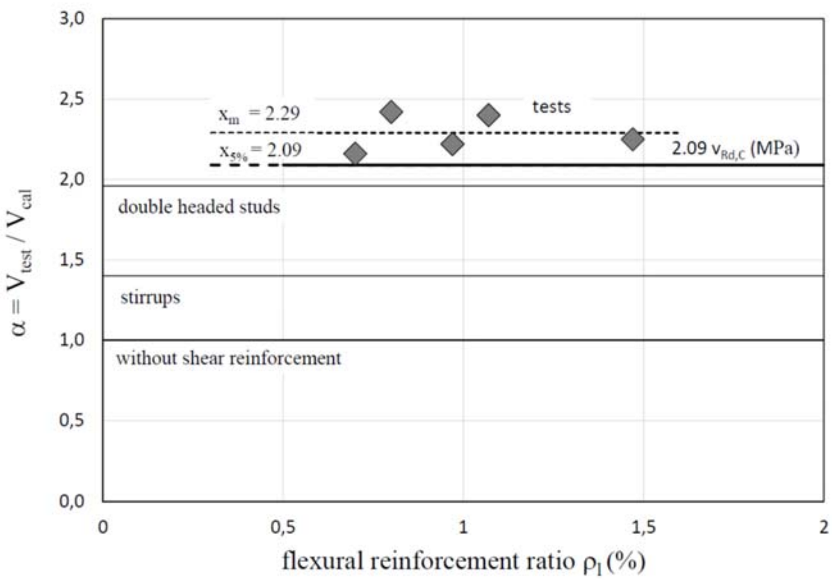


Figure 11 — Evaluation of the increasing factors according to DIN EN 1992-1-1/NA with eq. (1)

The design concept to determine the amount and the arrangement of this punching shear reinforcement is described in ETA-13/0521 and was proved by further testing in which a lower amount of shear reinforcement was chosen to ensure shear reinforcement failure (Siburg et. al (2014)).

EVALUATION ACCORDING TO ACI 318-14

According to the design concept of ACI 318-14, the punching shear strength must be proved in a critical section at a distance of 0.5d to the column. An inner column with a square cross section without openings or free edges nearby leads to a critical section as shown in Figure 10b. For critical sections for other boundary conditions, please refer to ACI 318-14. The stress corresponding to two-way shear strength v_c provided by normal concrete is given by equation (3). In contrast to eq. (1), there is no flexural reinforcement ratio or size effect taken into account. The nominal shear strength V_c is given by equation (4).

$$v_c = 1/3 \sqrt{f_c`} \quad \text{(MPa)} \tag{3a}$$

$$v_c = 4 \sqrt{f_c`} \quad \text{(psi)} \tag{3b}$$

v_c = stress corresponding to two-way shear strength provided by concrete
$f_c`$ = specified compressive strength of concrete

$$V_c = v_c \cdot b_o \cdot d \tag{4}$$

V_c = nominal shear strength provided by concrete

Table 1 provides the ratios of the tested shear resistance V_{test} to the expected shear strength V_{cal} according to eq. (4), taking into account the specified compressive strength of concrete $f_c`$ as the average value f_{cm} of the tested cylinders. The single values of V_{test}/V_c are between 2.15 and 2.55 with an average value of 2.41. The coefficient of variation 0.068 is higher than determined according to DIN EN 1992-1-1/NA. The higher scatter is a result of the varied flexural reinforcement ratios in the tests, which is not taken into account in eq. (3). The characteristic value is estimated as V_{test}/V_c = 2.12.

Figure 12 shows the single values in comparison with the nominal shear strength of slabs with and without shear reinforcement according to ACI 318-14. For stirrups and double headed studs with distances above 0.5d, ACI 318-14 stipulates a maximum shear resistance of 1.5 v_c according to eq. (3). For double headed studs with a maximum distance s = 0.5d, the maximum two way shear resistance is given by 2.0 v_c. Both the single values and the characteristic value for the tested lattice shear reinforcement are above the limit for slabs with double headed studs with small distances. Therefore the maximum two way shear resistance can be taken as for double headed studs according to equation (4).

$$v_u = 2/3 \sqrt{f_c`} \quad \text{(MPa)} \tag{4a}$$

$$v_u = 8 \sqrt{f_c`} \quad \text{(psi)} \tag{4b}$$

The shear resistance increases with the flexural reinforcement ratio. Two of the tests were carried out with a small flexural reinforcement ratio. An evaluation of the test only with a ratio of at least about 1% gives a characteristic increasing factor of 2.27. If flexural reinforcement of this amount is provided, a higher maximum two way shear resistance of $v_u = 0.75 \sqrt{f_c`}$ (MPa) or $v_u = 9 \sqrt{f_c`}$ (psi) respectively can be endorsed.

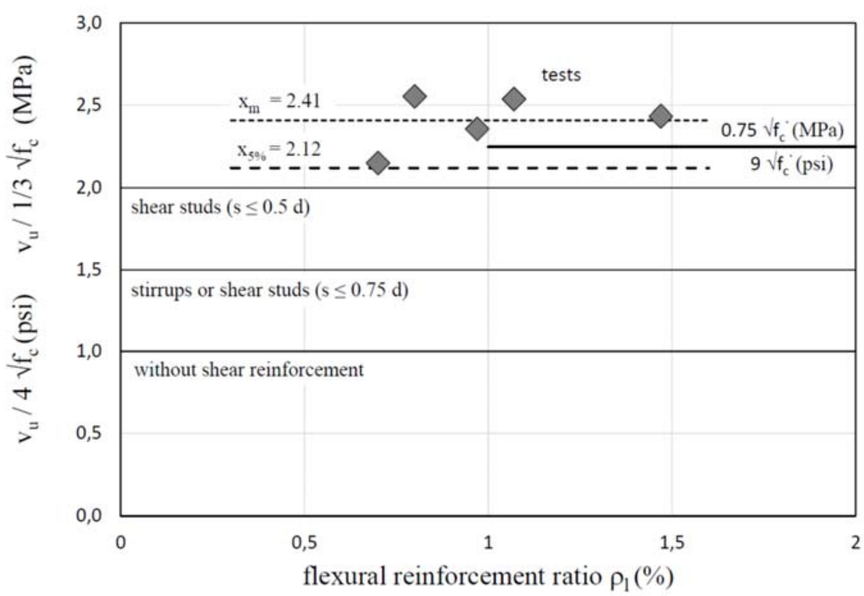


Figure 12 — Evaluation of the maximum test loads according to ACI 318-14 with eq. (3)

RESEARCH SIGNIFICANCE AND FURTHER STUDY

The reported research shows the effectiveness of the introduced special shaped lattice punching shear reinforcement. The application of this system increases the punching shear resistance of a flat slab more than double utilizing the easy installation of the reinforcement elements parallel to each other. It is already used successfully on the basis of the European Technical Approval ETA-13/0521. According to the major application the reported investigations concern gravity load.

In seismic regions the behavior of a slab-column connection under cyclic lateral loading is of particular interest. Park et al. (2012) tested lattice girder as punching shear reinforcement under cyclic lateral loading. The shape was quite different from the here introduced one and the flexural reinforcement in the tests was arranged on the inside of the lattice girder chords. However the lattice girders were also installed parallel to each other. Park et al. (2012) showed a high moment-carrying capacity and a high deformation capacity of the slab-column connection. This was proved also in the case where the lattice reinforcement was placed perpendicular to the lateral load direction. Similar tests with the here introduced filigran punching shear reinforcement could be useful to extend the application range for seismic regions.

SUMMARY

The maximum punching shear resistance of two-way slabs depends on the effectiveness of the punching shear reinforcement. Lattice girders with a stiff anchorage of the load bearing bars and a special geometry show good performance as punching shear reinforcement. Five full scale tests with column slab connections with different slab thicknesses, different column widths, different concrete strengths and different flexural reinforcement ratios were carried out. The failure mode was punching near the column. The 5% quantile of the shear resistance was 2.09 times higher than that of slabs without shear reinforcement and is approved according to DIN EN 1992-1-1/NA in ETA-13/521. An evaluation on the basis of ACI 318-14 yields a maximum shear resistance which is 2.12 times higher than that of slabs without shear reinforcement. Therefore the maximum two way shear strength can be taken as $v_u = 2/3\ \sqrt{f_c`}$ (MPa) or $v_u = 8\ \sqrt{f_c`}$ (psi) respectively. If flexural reinforcement ratios of at least 1% are provided, the resistance is higher.

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