New Punching Shear Reinforcement System for Footings and Ground Slabs

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Abstract. Punching shear tests on footings indicated that the inclination of compression struts is much steeper compared to flat slabs. The resulting steeper inclination of shear cracks leads to the assumption that vertically arranged punching shear reinforcement elements are less efficient in footings than in flat slabs. On that basis, a new punching shear reinforcement element with inclined bars was developed. At RWTH Aachen University, 14 punching shear tests on reinforced concrete footings including the new punching shear reinforcement elements were conducted. The test specimens partly failed inside the shear-reinforced zone and at maximum load level. In spite of a reduction of total steel cross sectional area, the test specimens with the new punching shear elements showed a significant increase in punching shear capacity (40–50%) compared to previous test series. The new punching shear reinforcement system allows for a significant reduction of footing's dimensions.

Keywords: Footings · Maximum punching shear capacity · Punching shear · Punching shear reinforcement · Shear span-depth ratio

1 Introduction

The punching shear behaviour of reinforced concrete footings without punching shear reinforcement has been investigated extensively by various researchers in the past (Talbot 1913; Richart 1948; Dieterle and Steinle 1981; Dieterle and Rostásy 1987; Hallgren et al. 1998; Hegger et al. 2006, 2007b, 2009; Siburg and Hegger 2014). According to these investigations, punching shear resistance mainly depends on the flexural reinforcement ratio, concrete compressive strength, and footing dimensions (e.g. effective depth, shear span-depth ratio, size effects), which is in line with previous investigations on flat slabs. However, due to more compact dimensions and soil-structure interaction, footings and ground slabs achieve significantly higher punching shear capacities than flat slabs (Hegger et al. 2006; Hegger et al. 2007b; Hegger et al. 2009; Siburg and Hegger 2014).

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Fewer experimental investigations have been conducted on reinforced concrete footings with punching shear reinforcement (Hegger et al. 2009; Siburg and Hegger 2014). The test results indicate that vertical punching shear reinforcement elements like stirrups (Beutel and Hegger 2002; Fernandez Ruiz and Muttoni 2009; Hegger et al. 2007a) and studs (Andrä 1981; Mokthar et al. 1985; Ricker and Häusler 2014; Ferreira et al. 2014) are less efficient in footings than in flat slabs. Due to the steeper inclination of shear cracks in footings, a higher efficiency of inclined punching shear reinforcement elements (e.g. inclined shearband reinforcement (Pilakoutas and Li 2003), lattice girders (Park et al. 2007), bent-up bars (Einpaul et al. 2016)) can be assumed. A punching test on a reinforced concrete footing with bent-down bars seems to confirm this assumption (Dieterle and Rostásy 1987).

Based on the results of previous test series (Dieterle and Rostásy 1987; Hegger et al. 2006, 2007b, 2009; Siburg and Hegger 2014), a new punching shear reinforcement system with inclined bars was developed. In a first test series (Kueres et al. 2017), the punching shear behaviour of footings with the new punching shear reinforcement element and a failure inside the shear-reinforced zone was investigated. Based on the results of the first test series, the maximum punching shear capacity of footings with the new punching shear reinforcement element was investigated in a second experimental campaign. A series of seven punching tests on reinforced concrete footings was conducted. All test specimens were provided with the new punching shear reinforcement.

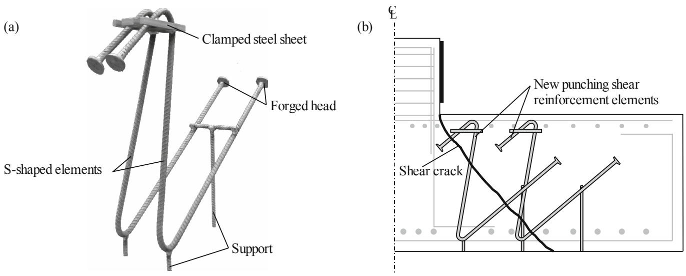

Fig. 1. New Punching shear reinforcement element

2 New Punching Shear Reinforcement Elements

The new punching shear reinforcement elements (Fig. 1 (a)) have an optimized form, which allows the shear crack widths to be efficiently controlled. Due to the inclination of the different sections, the S-shaped elements cross the shear cracks several times (Fig. 1 (b)). The rigid anchorage of the S-shaped elements by means of forged heads, as well as the effective upper anchorage, consisting of a clamped steel sheet, also contributes to the increased failure loads of the test specimens. Another advantage of the upper anchorage element is that no expensive welding is necessary. The mandrel diameter chosen for the new punching shear reinforcement is smaller than allowable according to DIN EN 1992-1-1 (2011) due to space reasons. It was shown by photomicrographs of the hooked bars that the reduced mandrel diameter does not lead to an increased number of micro-cracks. The full-scale tests of the previous test series (Kueres et al. 2017) confirmed the assumption that the increased concrete pressure due to the small mandrel diameter does not cause premature concrete failure adjacent to the looped anchorage elements.

3 Experimental Campaign

3.1 General

To investigate the punching shear behaviour of column bases with the new punching shear reinforcement system and to evaluate its efficiency, a total of 14 punching shear tests on reinforced concrete footings with uniform soil pressure were conducted. In a first experimental campaign, seven tests (diameter of punching shear reinforcement: $\varnothing_w = 10$ mm and $\varnothing_w = 12$ mm) were performed to investigate the punching shear behaviour of footings with the new punching shear reinforcement and a failure inside the shear-reinforced zone. The tests were planned considering the results of former test series on footings without and with stirrups as punching shear reinforcement (Hegger et al. 2009; Siburg and Hegger 2014). Based on the first experimental campaign, a second experimental campaign (seven tests, $\varnothing_w = 14$ mm) was conducted to investigate the maximum punching shear capacity of footings with the new punching shear reinforcement. The main parameters investigated in both experimental campaigns were the concrete compressive strength, the shear span-depth ratio, the column perimeter-depth ratio, the amount and layout of punching shear reinforcement. In the first experimental campaign, the effect of a longitudinal reinforcement in the compression zone (top reinforcement) was additionally investigated.

3.2 Materials

For all test specimens, commercial ready mixed concrete was used. The concrete mixture was designed to produce a 28-day target cylinder strength of $f_{c,cyl} = 25$ MPa and $f_{c,cyl} = 55$ MPa, respectively. For the lower concrete strength, ordinary CEM II 42.5 R Portland cement and a water-cement-ratio (w/c) of 0.65 to 0.73 was used, resulting in a slump of approximately 480 mm. For the concrete with higher strength, ordinary CEM I 52.5 R Portland cement and a water-cement-ratio (w/c) of 0.40 to 0.43 was used, resulting in a slump of approximately 470 mm. The maximum coarse aggregate size was 16 mm for both mixtures. To prevent premature failure, high-strength concrete with concrete compressive strengths between $f_{c,cyl} = 111.3$ MPa and 122.2 MPa was used for the column stubs. Additionally, the column stubs were strengthened with a steel collar made of 10 mm steel plates.

For all test specimens, the flexural reinforcement consisted of high-grade steel St 900/1100 with yield strengths varying from $f_y = 996$ MPa to 1044 MPa, a tensile strength of approximately $f_t = 1177$ MPa, and a Young's modulus of approximately $E_s = 194{,}300$ MPa. The high-grade steel was used to prevent a premature flexural failure. The new punching shear reinforcement elements were produced of steel B500 B, with measured yield strengths varying from $f_y = 553$ MPa to 585 MPa, tensile

Table 1. Test parameters

Specimen	h	d	c	b	$u_{0/d}$	$a_{\lambda/d}$	$f_{c,cyl}$	∅	f_y	ρ_l	top rft	$∅_w$	f_{yw}	Lay--out	V_{flex}	V_{test}
–	mm	mm	mm	mm	–	–	Mpa	mm	Mpa	%	–	mm	Mpa	–	KN	KN
Experimental campaign I																
DF_N1	450	399	200	1800	2.01	2.01	22.5	20	1044	0.79	yes	12	585	II	9862	4082
DF_N2	450	394	200	1800	2.03	2.03	22.6	20	1044	0.79	no	12	585	II	9872	4054
DF_N3	450	394	200	1800	2.03	2.03	20.5	20	1034	0.79	yes	10	547	II	9566	4544
DF_N4	450	394	300	1900	3.05	2.03	20.4	20	1034	0.83	yes	10	547	II	11,297	5045
DF_N5	450	396	300	1900	3.03	2.02	25.2	20	1034	0.83	yes	10	552	IIb	11,905	5937
DF_N6	450	397	400	2000	4.03	2.02	21.2	20	1037	0.86	yes	10	552	II	13,285	6515
DF_N7	450	396	300	1300	3.03	1.26	22.8	20	1037	0.85	yes	10	552	I	14,557	6573
Experimental campaign II																
DF_N8	450	397	300	1300	3.02	1.26	23.0	20	996	0.79	yes	14	553	I	14,020	7743
DF_N0 N	450	396	300	1900	3.03	2.02	22.4	20	1034	0.83	yes	14	569	II	11,437	6380
DF_N9	450	397	300	2700	3.02	3.02	25.1	20	996	0.81	yes	14	553	III	9928	5629
DF_N11	450	397	300	1900	3.02	2.02	53.8	20	996	0.83	yes	14	553	II	12,731	9831
DF_N12	450	397	300	2700	3.02	3.02	51.3	20	996	0.81	yes	14	553	III	10,977	7850
DF_N13	450	395	400	2000	4.05	2.03	22.8	20	996	0.86	yes	14	553	II	12,945	7416
DF_N14	450	396	600	2200	6.06	2.02	21.7	20	996	0.86	yes	14	553	II	15,713	8692

h: slab thickness; *d*: effective depth; *c*: square column dimension; *b*: square footing dimension; u_0/d: specific column perimeter; a_λ/d: shear span-depth ratio; $f_{c,cyl}$: concrete compressive strength; ∅: diameter of longitudinal reinforcement; f_y: yield strength of longitudinal reinforcement; ρ_l: longitudinal reinforcement ratio; $∅_w$: diameter of shear reinforcement; f_{yw}: yield strength of shear reinforcement; layout: layout of punching shear reinforcement; V_{flex}: shear force that produces flexural failure according to yield-line theory; V_{test}: ultimate failure load.

strengths in a range of f_t = 631 MPa and 646 MPa, and a Young's modulus between E_s = 199,100 MPa and 199,800 MPa. Table 1 summarises the properties of the materials used.

3.3 Test Specimens

The test series of the experimental campaign I consisted of seven reinforced concrete footings with side dimensions of 1300, 1800, 1900, and 2000 mm in square. The seven test specimens of the experimental campaign II had side dimensions of 1300, 1900, 2000, 2200, and 2700 mm in square. All specimens had a slab thickness of 450 mm. The square column stubs had side dimensions of 200, 300, and 400 mm (campaign I) and 300, 400, and 600 mm (campaign II) and were cast monolithically at the centre of the footing. The distance between the outer compression fibre and the centroid of the tension reinforcement (effective depth) was approximately d = 400 mm for all specimens, resulting in shear span-depth ratios between a_λ/d = 1.25 and 2.00 (with a_λ being the distance from the column face to the edge of the footing) for campaign I and between a_λ/d = 1.25 and 3.00 for campaign II. The specific column perimeter was in the range of u_0/d = 2.0 and 4.0 (campaign I) and u_0/d = 3.00 and 6.00 (campaign II). In both test series, the flexural reinforcement ratio varied between ρ_l = 0.79% and 0.86%. In the first experimental campaign, the diameter of the inclined bars of the punching shear reinforcement elements was either $\varnothing_w$ = 10 mm or 12 mm. In the second experimental campaign, the diameter of the inclined bars was 14 mm for all specimens. The different layouts of punching shear reinforcement investigated are shown exemplarily in Fig. 3 for test specimens DF_N0 N (Layout II) and DF_N9 (Layout III). Layout II consisted of eight punching shear reinforcement elements in the first row and eight elements in the second row. In the slender specimens (DF_N9 and DF_N12, a_λ/d = 3.00), a third row, consisting of eight punching shear reinforcement elements, was installed. Specimen DF_N8 (Layout I) was tested with a shear span-depth ratio a_λ/d = 1.25. Hence, only one row of punching shear reinforcement could be installed due to space limitations.

3.4 Test Setup and Measurements

The punching shear tests were conducted in accordance with former tests on reinforced concrete footings without and with punching shear reinforcement (Siburg and Hegger 2014). The specimens were tested upside down with the base area on top and loaded by a uniform surface load. The uniform soil pressure was simulated by means of 25 load application points. The load was applied depending on the expected failure load by either 13 hydraulic jacks (12 hydraulic jacks + 1 hydraulic jack with a piston area of half the size), which transferred their load through cross beams, or 25 hydraulic jacks (Fig. 2). All hydraulic jacks were linked to a common manifold and applied the same load independent of the displacement. In order to avoid any formation of membrane forces in the specimens, polytetrafluoroethylene-coated (PTFE) sliding and deformation bearings with dimensions of 140 × 140 mm were placed between the specimens and the cross beams or hydraulic jacks, respectively.

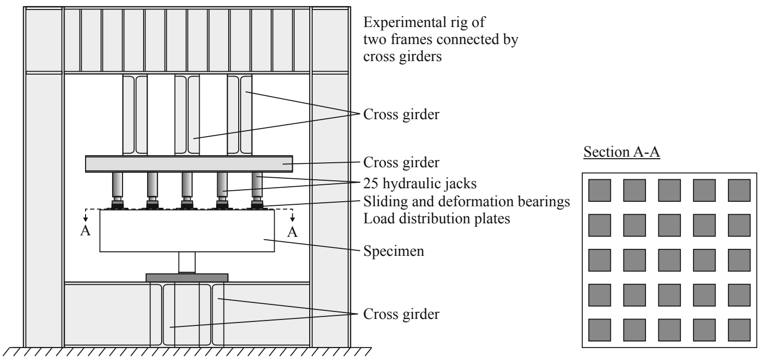

Fig. 2. Test setup

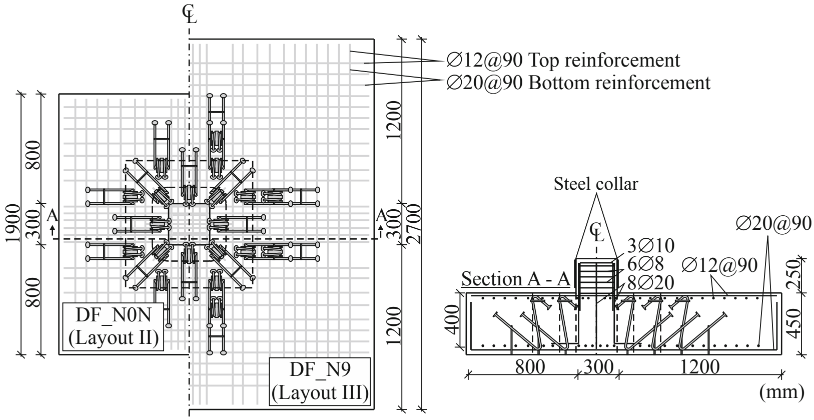

Fig. 3. Layout of flexural reinforcement and punching shear reinforcement for specimens DF_N0 N (Layout II) and DF_N9 (Layout III)

During testing, the vertical displacement of the test specimens was recorded at the corners of the column stub and at the footing's corners using linear variable differential transformers (LVDTs). To investigate the development of the inner shear cracks, the increase in the slab thickness was measured at several points and the penetration of the column into the slab was monitored. Strain gages were used to measure the strains in the flexural reinforcement at six locations and at several points in the punching shear reinforcement elements. To obtain the average strain at the bar's centre of gravity, two strain gages were attached to opposite side faces of the reinforcing bars at each measuring point. The concrete strains were recorded at four locations on the compression face of the footing near the column.

3.5 Test Procedure

The load was applied load controlled in increments of 200 kN (campaign I) or 400 kN (campaign II), respectively. To simulate lifetime loading, the load was cycled ten times between a calculated service load $V_{service}$ and half its value. For the specimens DF_N1, DF_N2, and DF_N3 (campaign I), the service load was 1200 kN corresponding to 40% of the predicted punching shear capacity of an identical footing without punching shear reinforcement according to DIN EN 1992-1-1 (2011) and DIN EN 1992-1-1/NA (2013). For the other specimens of the first experimental campaign, the service load was increased to 1400 kN. For the specimens of the second experimental campaign the service load was defined as the predicted failure load of an identical footing without punching shear reinforcement according to DIN EN 1992-1-1 (2011) and DIN EN 1992-1-1/NA (2013). After the load cycles, the specimens were continuously loaded until failure took place.

4 Experimental Results

4.1 Failure Characteristic

All tests failed in punching of the footing. The failure loads V_{test} are listed in Table 1. Before the failure occurred, increasing slab thickness, increasing strains in the punching shear reinforcement and penetration of the column stub into the slab were observed. The comparison with the flexural capacities of the footings V_{flex} according to yield-line theory (Gesund 1983) in Table 1 reveals the fact that the flexural capacities were not reached and hence confirms that failure occurred due to punching. Strain measurements verify this observation.

4.2 Cracking Characteristics

After testing, saw-cuts of the specimens were used to examine the inner shear crack patterns. In this context, Fig. 4 shows typical crack patterns of specimens with the new punching shear reinforcement and different punching failure modes.

All crack patterns showed finely distributed shear cracks crossing the punching shear reinforcement at several locations. Strain measurements confirmed that the punching shear reinforcement was activated. The saw-cuts of the specimens with a punching failure inside the shear-reinforced zone (e.g. DF_N5, campaign I, Fig. 4a) showed many inclined shear cracks, especially between the first and second row of punching shear reinforcement. In contrast, the footings with a punching failure on the level of the maximum punching shear capacity (e.g. DF_N0 N, campaign II, Fig. 4b) showed a steep failure crack, which developed between the column face and the lower bend of the punching shear reinforcement elements in the first row. In the slender specimens with shear span-depth ratios $a_{\lambda}/d = 3.00$ (e.g. DF_N9, campaign II, Fig. 4c) the failure crack developed from the third row of punching shear reinforcement, which indicates a failure outside the shear-reinforced zone.

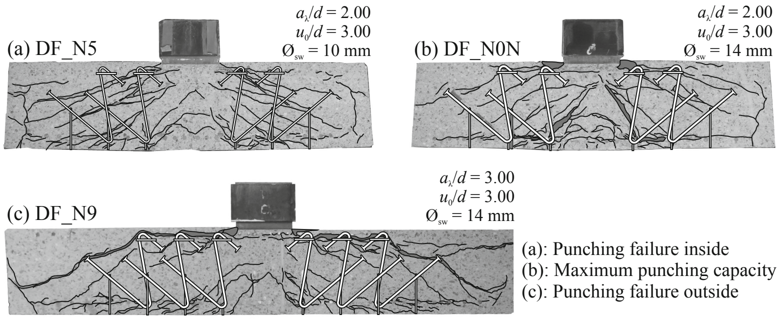

Fig. 4. Saw-cut of different specimens: Punching shear failure inside the shear-reinforced zone (a), punching shear failure at maximum load level (b), and punching shear failure outside the shear-reinforced zone (c).

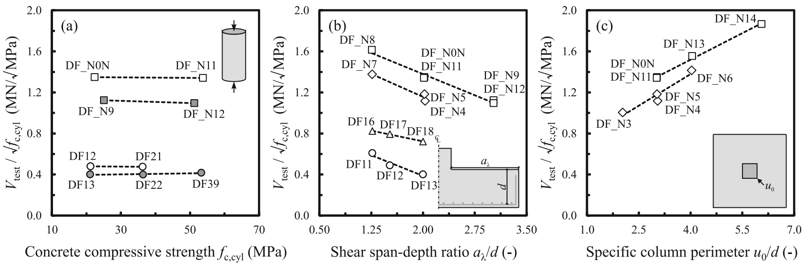

Fig. 5. Effects of concrete compressive strength $f_{c,cyl}$ (a), shear span-depth ratio a_λ/d (b), and specific column perimeter u_0/d (c) on punching shear strength of reinforced concrete footings.

5 Discussion of Experimental Results

5.1 Influence of Concrete Compressive Strength

The influence of the concrete compressive strength on the punching shear capacity of flat slabs was investigated by various researchers in the past (Elstner and Hognestad 1956; Regan 1986; Gardner 1990; Hallgren and Kinnunen 1996; Ramdane 1996). As a result of these investigations, different punching shear design provisions (e.g. ACI 318-14, *fib* Model Code 2010) account for this influence by the square root of the concrete compressive strength $\sqrt{(f_{c,cyl})}$. The influence of the concrete compressive strength on the punching shear capacity of footings without punching shear reinforcement was investigated in two test series (Hegger et al. 2009; Siburg and Hegger 2014). In this context, Fig. 5a shows the normalised failure loads $V_{test}/\sqrt{(f_{c,cyl})}$ for specimens DF12 (Hegger et al. 2009) and DF21 (Hegger et al. 2009) (a_λ/d = 1.50), as well as for specimens DF13 (Hegger et al. 2009), DF22 (Hegger et al. 2009), and DF39 (Siburg and Hegger 2014) (a_λ/d = 2.00). Regardless of the shear span-depth ratio, the tests show no trend with increasing concrete compressive strength, which confirms the approach of $\sqrt{(f_{c,cyl})}$ for the presented footings.

To verify this effect for footings with the new punching shear reinforcement elements, specimens DF_N0 N and DF_N11 ($a_\lambda/d = 2.00$), as well as DF_N9 and DF_N12 ($a_\lambda/d = 3.00$), can be considered (Fig. 5a). As already observed for footings without punching shear reinforcement, the approach of the square root of the concrete compressive strength allows for a realistic description of the punching shear capacity of footings with the new punching shear reinforcement elements.

5.2 Influence of Shear Span-Depth Ration

In previous test series (e.g. Hegger et al. 2006, 2007b, 2009; Siburg and Hegger 2014), the effect of the shear span-depth ratio a_λ/d on the punching shear resistance of footings without and with stirrups as shear reinforcement was investigated. Figure 5b shows the normalised failure loads $V_{test}/\sqrt{(f_{c,cyl})}$ for specimens DF11 ($a_\lambda/d = 1.25$), DF12 ($a_\lambda/d = 1.50$), and DF13 ($a_\lambda/d = 2.00$) without punching shear reinforcement as well as specimens DF16 ($a_\lambda/d = 1.25$), DF17 ($a_\lambda/d = 1.50$), and DF18 ($a_\lambda/d = 2.00$) with stirrups as punching shear reinforcement. The shear-reinforced specimens failed at maximum load level, which was indicated by the measured steel strains far below the corresponding yield strain (Hegger et al. 2009). The tests indicate that the punching shear resistance decreases with increasing shear span-depth ratio. This influence is less pronounced for footings with stirrups as punching shear reinforcement than for footings without punching shear reinforcement. Thus, the efficiency of stirrups decreases with decreasing shear span-depth ratio.

To verify this effect for the footings with the new punching shear reinforcement elements, specimens DF_N7 ($a_\lambda/d = 1.25$), DF_N4 ($a_\lambda/d = 2.00$), and DF_N5 ($a_\lambda/d = 2.00$), with a failure inside the shear-reinforced zone (campaign I), as well as specimens DF_N8 ($a_\lambda/d = 1.25$), DF_N0 N ($a_\lambda/d = 2.00$), DF_N11 ($a_\lambda/d = 2.00$), DF_N9 ($a_\lambda/d = 3.00$), and DF_N12 ($a_\lambda/d = 3.00$) of the second experimental campaign can be considered (Fig. 5b). All these specimens were tested with a column perimeter-depth ratio $u_0/d = 3.00$. In contrast to the test results of the previous test series (Hegger et al. 2009), a similar correlation between punching shear resistance and shear span-depth ratio as for footings without punching shear reinforcement can be observed regardless of the failure mode. Hence, the efficiency of the new punching shear reinforcement with inclined bars seems not to be affected by the shear span-depth ratio.

5.3 Influence of Column Perimeter-Depth Ratio

Previous experimental investigations on the punching shear behaviour of flat slabs showed a strong correlation between the punching shear resistance and the column perimeter-depth ratio u_0/d (e.g. Regan 1986, 2004). For small ratios u_0/d, the punching shear resistance decreases, which is for example taken into account in the code provisions of DIN EN 1992-1-1 (2011) and DIN EN 1992-1-1/NA (2013) or a new uniform design method for punching shear in flat slabs and column bases (Kueres et al. 2017).

A verification of this effect for footings by means of the results of the previous test series (Hegger et al. 2006, 2007b, 2009; Siburg and Hegger 2014) on footings without and with stirrups as punching shear reinforcement is not possible, since the tests were conducted with a uniform u_0/d-ratio of 2.0.

Figure 5c shows the normalized failure loads $V_{test}/\sqrt{(f_{c,cyl})}$ for specimens DF_N3 (u_0/d = 2.00), DF_N4 (u_0/d = 3.00), DF_N5 (u_0/d = 3.00), and DF_N6 (u_0/d = 4.00), with a failure inside the shear-reinforced zone (campaign I), as well as for specimens DF_N0 N (u_0/d = 3.00), DF_N11 (u_0/d = 3.00), DF_N13 (u_0/d = 4.00), and DF_N14 (u_0/d = 6.00) of the second experimental campaign. Regardless of the failure mode, the tests indicate an increasing punching shear resistance with increasing column perimeter-depth ratio.

6 Conclusions

The results of the experimental investigations on reinforced concrete footings with the new punching shear reinforcement system allow the following conclusions to be drawn:

- The new punching shear reinforcement elements with inclined bars significantly increase the punching shear capacity of reinforced concrete footings. The high efficiency of the new punching shear reinforcement is also evident for footings with higher concrete compressive strength.
- Due to the rigid anchorage and the optimized geometry of the new reinforcement elements, the punching shear capacity of reinforced concrete footings can be considerably increased compared to similar footings with stirrups.
- Many code provisions account for the influence of the concrete compressive strength on the punching shear capacity by the square root of the concrete compressive strength. This approach could be verified by the tests on reinforced concrete footings with the new punching shear reinforcement element.
- While punching shear tests on reinforced concrete footings with stirrups indicate a reduced efficiency of the shear reinforcement with decreasing shear span-depth ratio, the efficiency of the new punching shear reinforcement elements seems not to be affected by the shear span-depth ratio.
- Regardless of the failure mode, the conducted tests on reinforced concrete footings with the new punching shear reinforcement indicate an increasing punching shear resistance with increasing column perimeter-depth ratio.
- The new punching shear reinforcement system allows for a significant reduction of footing's dimensions. Another advantage could be the reduction of minimum reinforcement in ground slabs due to reduced slab thicknesses.

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