PUSH–OUT EXPERIMENTS OF HEADED SHEAR STUDS IN GROUP ARRANGEMENTS

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ABSTRACT: Precast concrete slabs connected to the steel beams by headed shear studs arranged in groups may be used in high-rise office buildings to speed up construction process and improve quality by increasing the grade of industrialization. The paper presents experimental research performed on push-out tests at the Faculty of Civil Engineering University in Belgrade. The main goal has been to investigate possible reductions (if any) of shear stiffness and strength of five group arrangements consisting always of four headed shear studs, the shank diameter 16 mm and 100 mm height. A shape of group arrangement and its orientation to the applied load, and distance between the studs were variables considered. Intention was to have as small as possible holes in precast concrete slabs so the distance between studs was lower than minimum required in Eurocode 4 [1]. Comparison is made with results obtained on specimen with uniformly distributed shear studs, according to requirements of the standard arrangement [1]. Material characteristics were obtained and push-out tests were preformed in procedures that fully comply with Eurocode 4.

Experimental results were evaluated according to the design models in Eurocode 4 and available literature, Okada-Yoda-Lebet [2]. Recommendations for use of the group arrangements were proposed.

Keywords: Headed studs, Group arrangement of stud connectors, Steel-concrete composite beams, Prefabricated concrete slab

1. INTRODUCTION

Devices used to establish transfer of longitudinal shear force between the steel beam and concrete slab strongly influence structural characteristics of a composite structure. In composite floors of high-rise buildings, the headed shear studs welded to a steel flange and encased in concrete is the most common way to insure composite action. Continuous and uniform distribution of headed studs on the flange of the steel beam results in considerable limitations for use of prefabricated composite floors. Groups of headed studs placed at the predefined positions matching openings of the prefabricated reinforced concrete slab are often used for insuring the effective composite action. In Eurocode 4, for the composite bridges EN1994-2 [3] design of discontinuously placed headed studs is allowed. A group of headed studs is allowed in the most heavily loaded zones of beams (close to the supports).

If the recommendation of the minimum spacing between the individual headed studs (5*d* in the force direction) is strictly considered, a larger number of grouped shear studs would require larger dimensions of openings in the prefabricated slab. The minimum allow space between the headed studs in the direction perpendicular to the force (2,5*d*) is practically equal to the minimum spacing due to the welding. This is defined by the dimensions of equipment for automatic welding. For this reason, a possibility of reducing the distance between the headed studs in the force direction is analyzed.

Shim et al. [4] analyzed the shear resistance of the group of 9 headed studs 25 mm in diameter as a function of the distance between the studs which are smaller than the stipulated distance of the 5*d*. Josef Hegger et al. [5] considered shear resistance of two headed studs placed at the distance of 2*d*, in the force direction and a comparison with the shear resistance of specimens with two studs at the distance 2,5*d* was presented. For the sake of comparison, in experiments shown in this paper the concrete with lower strength and headed studs with the higher ratio h_{sc}/d are used.

Shim et al. [4] propose reduction for the shear resistance of studs in a group compare to the sum of shear resistance of individual studs. This reduction of 30% for a group of headed studs at a distance of 3*d* in the force direction is certainly a function of the distance between them. However, Hegger et al. [5] presented results of the experiment where they concluded that the shear resistance of two headed studs at the distance of 2*d* in the force direction was only 5.4% lower than the sum of individual shear resistances. Hegger et al. [5] used high strength concrete which gave smaller reduction of the shear resistance. Okada et al. [2] analyzed group of 9 studs with different distances between studs (from 3*d* to 13*d*) in force direction and with various concrete strength. The interaction of reinforcement layout and shear resistance was experimentally analyzed, as well. In our experiments, the specimens were made of the standard concrete strength.

Load-slip characteristic of single headed stud in the shear connection is rather well known and described in literature, [6] and [7], but there are no many data on load-slip characteristics of the group of studs. No information and recommendations on the relationship between the load-slip characteristic of individual headed stud and the load-slip characteristic of the group of studs with nominally identical resistance are available.

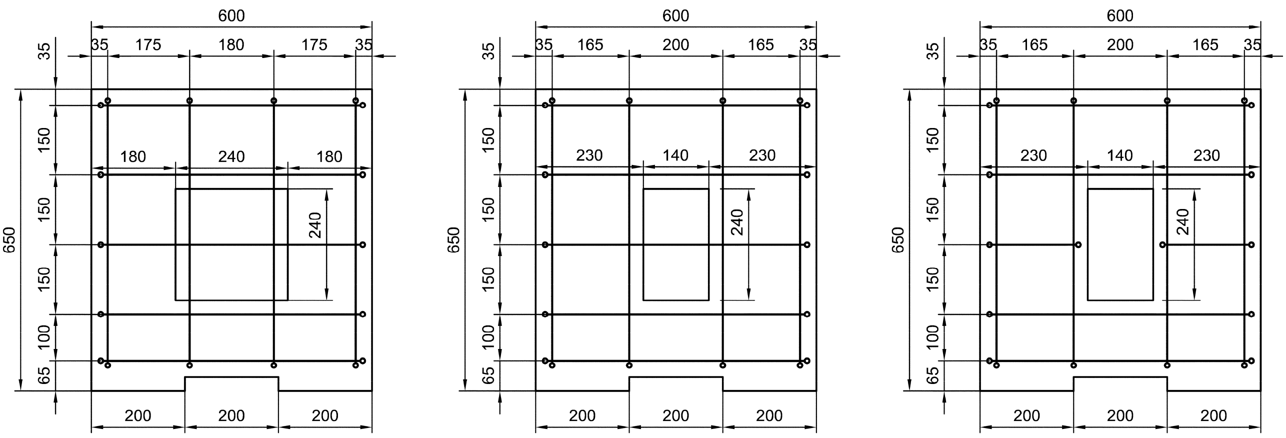
The shear resistance of studs is dependent on several parameters, such as: the stud material, diameter of the stud, strength and elasticity modulus of concrete in which the stud is installed. Experiments on the shear resistance of an individual stud, which have been conducted for decades, have demonstrated that the strength of concrete, height of headed studs, i.e. height/diameter ratio of studs (h_{sc}/d) are the most influential parameters for the shear resistance. Current design codes [1], [3], [8] take into account the effects of height/diameter ratio (h_{sc}/d) explicitly on the shear resistance.

This paper provides information on experimental part of the ongoing research project on headed shear stud behavior in a composite structure with a prefabricated concrete slab.

The main goal of these experiments is to investigate potential of reducing spacing between the headed studs, less than the minimum recommended value of 5*d* in the force direction. The specimens with groups of 4 headed studs d=16 mm in diameter and the height h_{sc}=100 mm have been used. In total, 25 specimens have been tested on the standard push-out test. The arrangement of headed studs within a group, as well as the orientation of the group to the direction of the load action, have been varied.

2. TEST SET-UP

The specimens included in this research are made and tested in accordance with the Annex B of Eurocode 4. They consist of: two prefabricated reinforced concrete slabs and the steel beam HEB260 with groups of headed studs welded by standard automatic procedure (Drawn Arc Stud Welding). Three types of prefabricated slabs have been used (Figure 1). All concrete slabs have standard dimensions, 600x650 mm, and they have openings for accommodating the groups of headed studs, which are located almost in the middle of the slab. Slab Type 1 has a standard dimensions acc. to the Annex B of the Eurocode4 [1] and the opening 240x240 mm. Type 2 is modified, a reduced size of the opening to 140x240 mm (Figure 1b). Type 3 has same dimensions (the same opening) as the Type 2, but without transversal reinforcement passing through the opening (Figure 1c). This modification is done to analyze possible effect of the reinforcement on the behavior of the group of studs in the shear connection. All slabs are constructed with concrete of the C30/37 class and reinforced by the ribbed reinforcement ∅10 mm of the class B500. The production of precast concrete slabs was realized in precast concrete elements workshop. Three standard cubes 150 mm were used for compressive strength test for each concrete mixture. Tension strength was determined by bending test on prisms100x100x400mm, also for each concrete mixture.



a) Type 1 *a) Type 2* *a) Type 3*

Figure 1. Types of Reinforced Concrete slabs

Specimens were casted in the laboratory. After assembling of prefabricated RC slabs into precisely defined positions, the slab openings were filled in. The in-fill material was concrete with strength ranging from f_{ck}=32-40MPa, and with small shrinking dilatation. Mix proportion for infill concrete is presented in Table 1. The water-cement ratio was 0.52. For infill concrete, three grain size of river aggregate and two admixtures were used (shrinkage-reducing and super-plasticizer) as presented in Table 1.

Table 1. Mix Proportion for Infill Concrete

Mixture components [kg/m3]						
Cement 42,5R	Water	Sand	Rounded (River) gravel aggregate		Admixtures	
		0-4	4-8	8-16	shrinkage-reducing	super-plasticizer
320	162	822	478	611	6.4	1.92

According to Eurocode 2, the maximum diameter of an aggregate is in relation with the minimum clear distance between rebar (in this case headed studs). The minimum clear distance between headed studs is 29 mm. For the development of an adequate bond between the concrete and rebar, and satisfactorily placed and compacted concrete between headed studs, the maximum nominal aggregate size 16mm was adopted.

Using concrete shrinkage reduction admixtures, 20% smaller shrinkage dilatation have been obtained than the design values according to the Eurocode 2 [9], see Figure 2. By measuring the shrinkage on the control prisms, shrinkage dilatation of 250x10-6, have been obtained for the age of concrete of 45 days at the air temperature of 20-25 °C and the relative humidity of 23±2%.

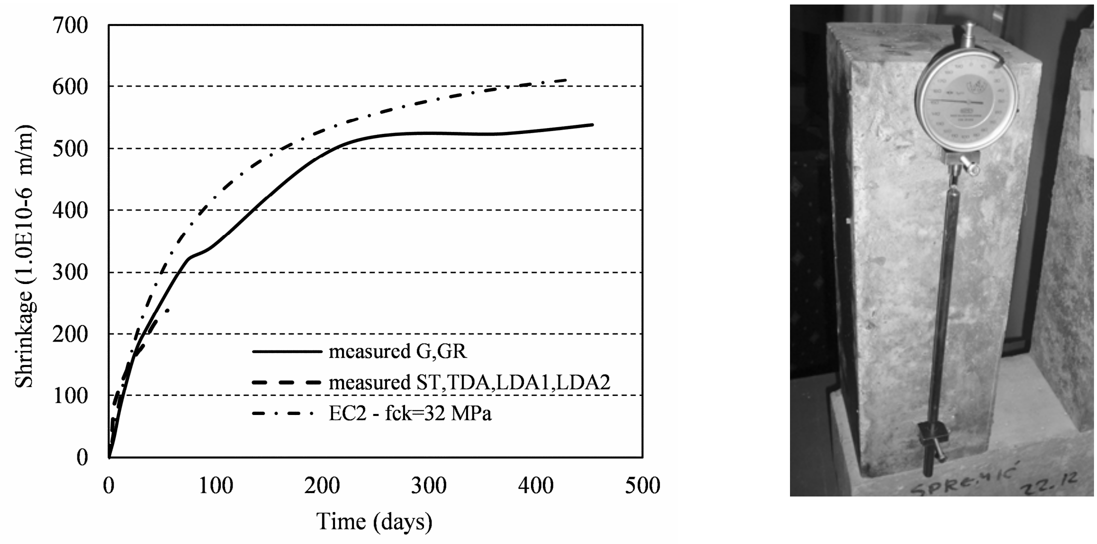


Figure 2. a) Shrinkage dilatation of the concrete in time, b) The Test Layout

Okada et al. [2] has analyzed the bond-friction effects between steel section and concrete slab. Their conclusion is that about 10% of total shear resistance of headed studs is achieved by bond-friction stresses.

The unreliable and brittle failure mode of the chemical bond between steel and concrete are reasons not to take the bond-friction stresses in design of composite beam. In our research, the chemical bond force between the concrete and steel beam was eliminated, as suggested in Eurocode 4 [1] in B.2.3(2). The surface of the steel beam was oiled prior to the assembling, to prevent chemical bond between the concrete and the steel beam. Just prior to the monolitization, surfaces of prefabricated slabs, which are in direct contact with fresh infill concrete, have been painted with the coating for contact joints between old and fresh concrete. Filling of concrete was performed in a position as shown in Figure 3. Three days after concreting of one side of the specimen, the specimens were reversed, and the other side of the specimens was completed. The infill concrete achieved full design strength 14 days after concreting (Figure 4). The samples are tested 21 days after placing concrete on the reverse side, meaning 24 days after placing concrete on the first side of the specimen. From the concrete strength diagram as the function of the concrete age (Figure 4) it can be seen that no significant increase of the concrete strength occurred after 21 days.

Figure 3. The Slabs Prior to Assembling

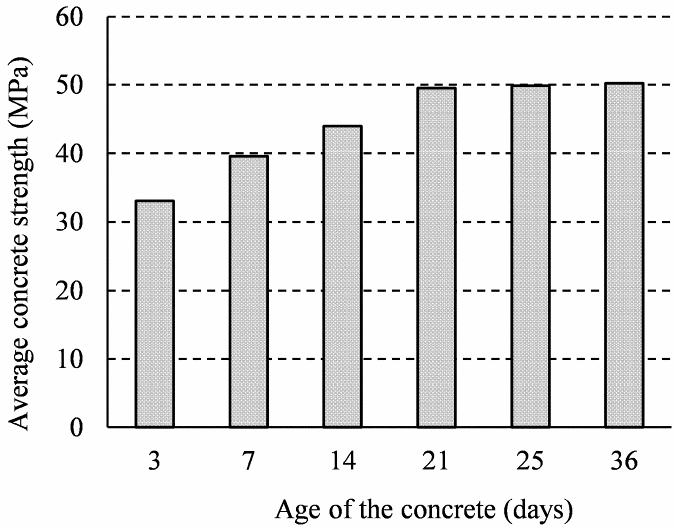
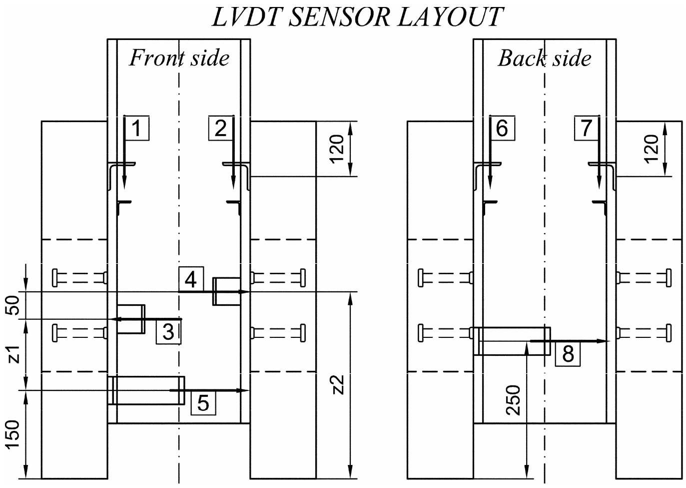
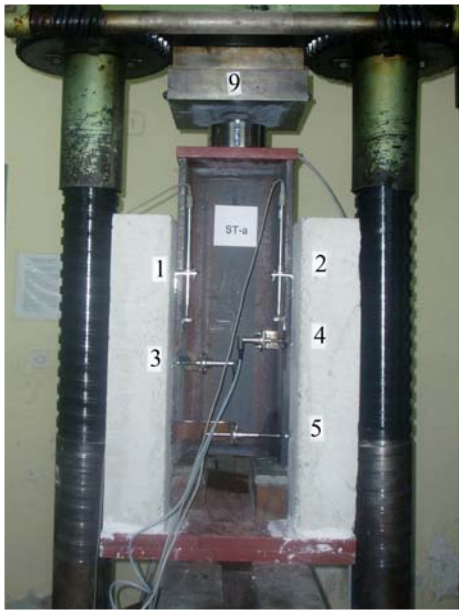


Figure 4. Increase of the Concrete Strength

The vertical force was applied by the hydraulic jack having the capacity of 3500 kN, in displacement control. In the first phase of testing, the specimens were loaded with cyclic loading. The load was applied in 26 cycles. In the first cycle, the load was applied incrementally, in three steps from 90 kN to the force of 270 kN, which is around 40% of the expected value at failure. Then the specimen is unloaded to 32 kN. The remaining 25 cycles comprised of loading of the specimen to 270 kN and it's unloading to 32 kN. After the last cycle of the first phase, the load was applied continually until failure, ensuring that specimen failure occurs 20-25 minutes since the start of load application in the last cycle. In order to reduce eccentricity, and to provide good contact of the specimen and the jack, the specimen was placed into the layer of fresh gypsum prior to mounting on the press. The jack headed is equipped by a spherical bearing. The force was monitored via the control board of the hydraulic jack and with the aid of load cells for measuring pressure force, manufactured by "Hottinger" having measuring range up to 1000 kN. During the test, slip of the steel beam in respect to the concrete slabs was monitored in four points, two points per each concrete slab, by the Linear Variable Displacement Transducer (LVDT sensors) no. 1, 2, 6, 7 from the Figure 5a. The transversal deflection – separation of the concrete slabs from the steel beam was also monitored in two points (LVDT sensors no. 3 and 4). Positions of the measuring points 3 and 4 are approximately at the location of the stud group. Horizontal separation between concrete slabs was monitored too. One sensor for monitoring of the slabs separation was placed at the location of the stud's group (sensor 8) while the other one, sensor number 5, was placed at 150 mm from the lower edge of the specimen. For LVDT sensor layout see Figure 5a. The deflections were monitored by inductive deflectometers "Hottinger". Data acquisition was performed continually by MGC+ "Hottinger" with the data recording frequency of 0,5 Hz and 1,0 Hz.





1,2,6,7 – LVDT sensors for longitudinal slip
3,4 – LVDT sensors for transverse separation between the steel section and concrete slab
5,8 – LVDT sensors for transverse separation between the concrete slabs

Figure 5. a) Layout of Measuring Points, b) Equipped Specimen Ready for Testing

3. THE GROUP ARRANGEMENTS

As the initial data for the analysis of shear resistance of the group of elastic headed studs, the shear resistance of the single headed stud was adopted. The shear resistance of headed stud having diameter d=16mm and the height h_{sc}=100 mm as the function of concrete strength is presented by the diagram of the Figure 6.

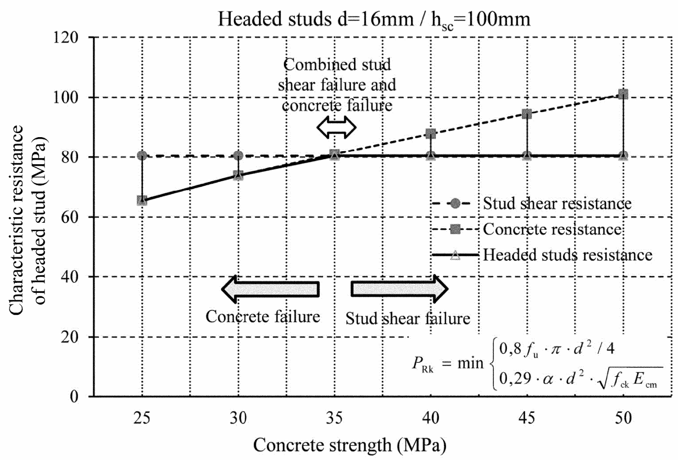


Figure 6. Resistance of Headed Studs as the Function of Nominal Concrete Strength [1]

Analyzing characteristic shear resistance of headed studs as the function of the concrete strength, (Figure 6), three failure modes can be observed: the shear stud failure, the concrete failure, and a combined failure mode. The combined failure mode is characterized by the crushing failure of concrete in the zone around the studs along with the shear failure of headed studs. If the characteristic strength of concrete has lower values (f_{ck}<35MPa), the failure of the concrete around the headed studs is expected. For the characteristic concrete strength of around f_{ck}= 35MPa, concrete failure around the connector and stud shear failure are approximately equal, so the combined failure is expected, while for the concrete strengths higher than f_{ck}= 40MPa stud shear failure can be expected.

To determine the expected shear resistance of a group of headed studs, the mechanical properties of the headed stud material were examined, the properties of the concrete used for monolitization of specimens. A characteristic stress-strain diagram of a shear stud was obtained using the coupon test, see Figure 7. Mechanical properties of concrete are given in Table 2.

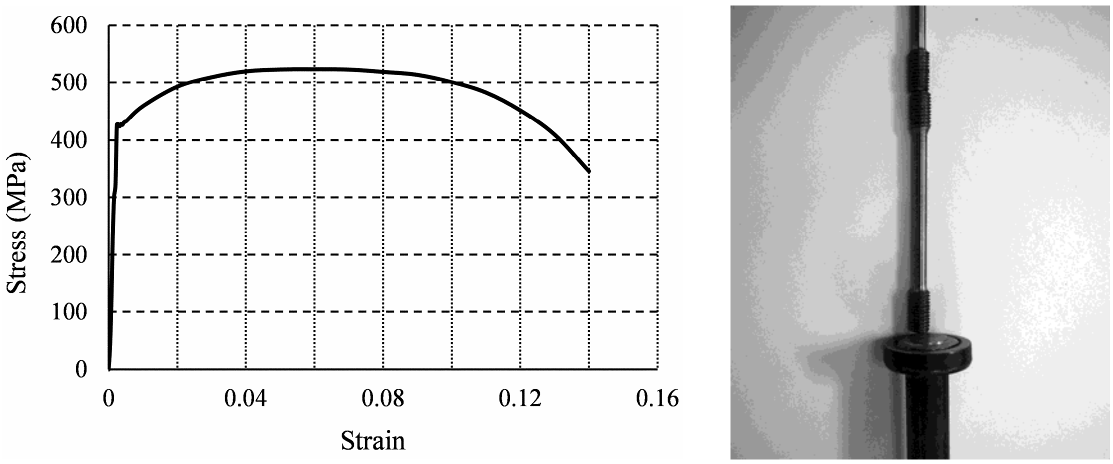
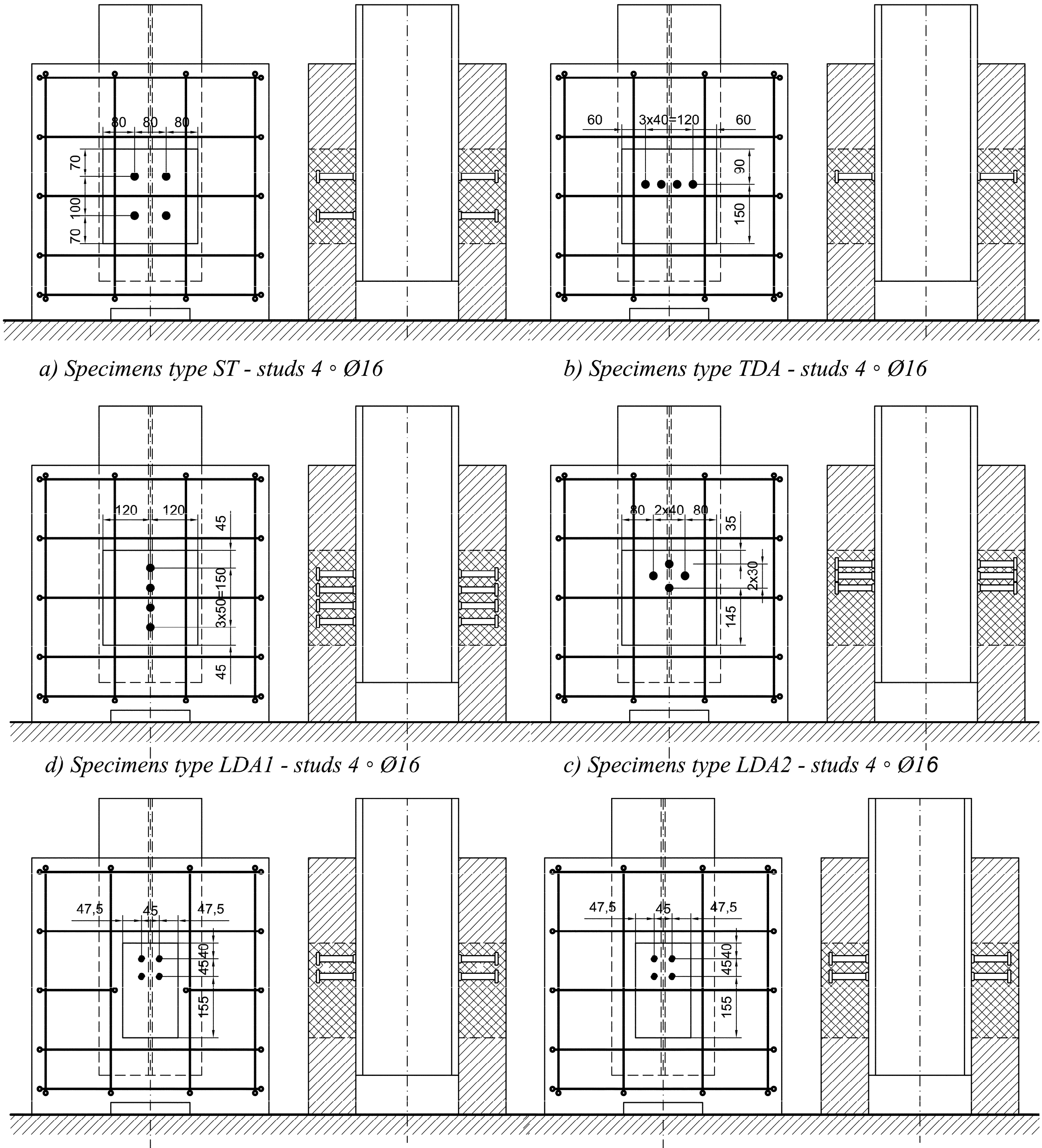


Figure 7. Characteristic Stress-strain Diagram for the Headed Stud Material

Experiment study included a total of 5 arrangements of studs. All the specimens were constructed with a total of 8 studs, four studs on each side. In the first phase of testing, four different layouts of headed studs groups were constructed. The standard specimen ST (standard test) consists of the group of four studs, 16 mm in diameter, on one side of the specimen. It has the arrangement which is fully according to Eurocode 4 (Figure 8a). The TDA (transversal distance arrangement) specimen types (Figure 8b) include the case when the same number of headed studs is placed in one row transversally in relation to the direction of the load, with the distance equal to the minimum distance [1] of 40 mm (2,5*d*). In the group with LDA1 (longitudinal distance arrangement - type 1) specimens, there are four studs in the group, also placed in one row along the direction of the load, at a distance of 50 mm (≈3*d*), which is less than the required minimal distance between the headed studs according to design codes (EC4). The last group specimens - LDA2 (longitudinal distance arrangement -type 2) was constructed using the rhomboid arrangement of four studs. The distance in the direction of the load is smaller than with those stipulated by design codes (Figure 8c).

The second phase of research included two new layouts of specimens: type G1 (group arrangement without reinforcement) (Figure 8e) and type GR1 (group arrangement with reinforcement) (Figure 8f). Since obtained values of the shear resistance of specimens in the first phase of the experiment were higher than expected, the distance between studs were reduced to 45 mm (≈2,5*d*) in the second phase. Type G1 specimens were formed with Type 3 concrete slabs (Figure 1c) with no reinforcement in the opening. GR1 specimens were formed with Type 2 concrete slabs. Considering that the headed stud shear failure was relevant for all the tested specimens in the first phase, the specimens in the second phase were monolitized with a slightly lower strength concrete instead of one having f_{ck}=40MPa (see Table 2). In the second phase, it was also analyzed to what extent the reinforcement in the opening affects the shear resistance of the headed studs group.



a) Specimens type ST - studs 4 ◦ Ø16 *b) Specimens type TDA - studs 4 ◦ Ø16*

d) Specimens type LDA1 - studs 4 ◦ Ø16 *c) Specimens type LDA2 - studs 4 ◦ Ø16*

e) Specimens type G1 - studs 4 ◦ Ø16 *f) Specimens type GR1 - studs 4 ◦ Ø16*

Figure 8. Layout of Specimens

4. TEST RESULTS

An overview of experimental results is shown in Table 2. In the column (2) and (3) are presented the strengths of infill concrete of the front/back side of the specimen. The average value of maximum slip at failure δ_{max}, between the concrete slab and steel flange, is presented in the column (7). In the column (8) is presented slip which corresponding the force value of 500kN. The force of 500kN is slightly higher than the force which corresponds to the service load (limit state of serviceability). The stiffness of shear connection k_{sc} (for eight studs) is also presented in the table for all 25 tested specimens (9). Column (10) of Table 2 presents the ratio of measured shear resistance of specimens (F_u) and characteristic design shear resistance in accordance with [1], which was determined on the basis of actual experimental characteristics of the material $F_{Rk,exp}$.

$$F_{Rk,\exp} = 8 \cdot \left[0{,}29 \cdot 1{,}0 \cdot (16\mathrm{mm})^2 \cdot \sqrt{f_{\mathrm{ck}} \cdot E_{\mathrm{cm}}}\right] \leq 8 \cdot \left[0{,}8 \cdot 523\,\mathrm{MPa} \cdot \frac{\pi \cdot (16\mathrm{mm})^2}{4}\right] \tag{1}$$

Table 2. Shear Resistance of Specimens

Specimen	$f_{c,cube,infill}$	$f_{ck,infill}$	$f_{ck,plate}$	E_{cm}	F_u	δ_{max}	δ (*500 kN*)*	k_{sc}	$F_u/F_{Rk,exp}$
	[MPa]	[MPa]	[MPa]	[MPa]	[kN]	[mm]	[mm]	[MN/mm]	[-]
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
ST a	47.6 / 47.6	39.6	30.7	36800	726.8	6.10	0.64	0.995	1.08
ST b	47.6 / 47.6	39.6	28.9	36800	737.0	5.80	0.47	1.308	1.10
ST c	47.6/50.0	40.8	30.7	36800	770.8	6.60	0.52	1.204	1.15
TDA a	49.0 / 50.0	41.5	30.7	36800	735.6	7.76	0.52	1.188	1.09
TDA b	48.0 / 49.3	40.65	30.7	36800	764.7	7.27	0.45	1.363	1.14
TDA c	47.6 / 49.3	40.65	30.7	36800	762.2	8.22	0.49	1.219	1.13
LDA1 a	49.0 / 47.6	40.3	28.9	36800	799.4	13.0	0.60	1.276	1.19
LDA1 b	48.0 / 49.3	40.65	28.9	36800	784.7	10.7	0.52	1.400	1.17
LDA1 c	48.0 / 50.0	41.0	30.4	36800	686.1	9.0	0.68	1.142	1.02
LDA1 d	39.3 / 42.0	32.65	33.1	34200	722.5	8.16	0.74	1.077	1.15
LDA1 e	39.3 / 42.0	32.65	33.1	34200	643.2	6.86	0.75	1.121	1.02
LDA2 a	47.6 / 47.6	39.6	28.9	36800	744.5	8.25	0.570	1.045	1.11
LDA2 b	47.6 / 49.3	40.65	28.9	36800	757.6	6.98	0.520	1.202	1.13
LDA2 c	47.6 / 50.0	40.8	30.4	36800	758.3	5.94	0.490	1.302	1.13
G1 a	39.3 / 42.0	32.6	29.8	34200	769.2	13.9	0.56	1.137	1.23
G1 b	39.3 / 43.0	33.2	29.8	34200	740.2	12.5	0.57	1.137	1.17
G1 c	39.3 / 42.0	32.6	29.8	34200	750.9	11.2	0.60	1.059	1.20
G1 d	39.3 / 43.0	33.2	29.8	34200	728.0	9.78	0.63	1.013	1.15
GR1 a	39.3 / 42.0	32.6	34.2	34200	758.0	12.5	0.64	0.973	1.21
GR1 b	39.3 / 43.0	33.2	34.2	34200	759.6	13.2	0.65	0.927	1.20
GR1 c	39.3 / 42.0	32.6	34.2	34200	752.0	/	0.53	1.127	1.20
GR1 d	39.3 / 43.0	33.2	34.2	34200	789.9	12.8	0.55	1.100	1.25
*) displacement at force of 500 kN									

Characteristic value of shear resistance for a group of studs have been determined using Annex B of Eurocode 4 [1], for series of three specimens, and in accordance with Annex D of Eurocode 0 [10], for series of four specimens.

If the deviation of individual measured parameter in respect to the average value is not higher than 10%, the characteristic value of shear resistance can be determined according to the B.2.5 of Annex B. The characteristic shear resistance of the specimen is equal to the minimum shear resistance of all the tested specimens reduced for 10%:

$$F_{stat1,Rk} = 0.9\, F_{u,min} \tag{2}$$

The characteristic values $F_{stat1,Rk}$ are shown in column (5) of Table 3. In column (6) of Table 3, the characteristic shear resistance $F_{stat2,Rk}$ according to the Annex D of Eurocode 0 are presented. The characteristic values $F_{stat2,Rk}$ of shear resistance according to the Eurocode 0, Annex D is:

$$F_{stat2,Rk} = m_X\,(1\text{-}k_n V_X) \tag{3}$$

where are:
m_X average value,
k_n according to table D1 in [10] (for 4 specimens k_n=2,63),
$V_X=s_X/m_X$ coefficient of variation (s_X is estimated value of standard deviation).
The characteristic values for series of three specimens obtained by Annex D and Annex B are almost equal.

Table 3. Characteristic Shear Resistance of the Group of Headed Studs

Specimens	F_{av}	δ_{av}	$F_{u,min}/F_{u,max}$	$F_{stat1,Rk}$	$F_{stat2,Rk}$	$F_{Rk,exp}$
	[kN]	[mm]	[-]	[kN]	[kN]	[kN]
(1)	(2)	(3)	(4)	(5)	(6)	(7)
ST f_{ck}=40 MPa	370.6	6.1	0.94	327.0		336.5
TDA f_{ck} =40 MPa	376.4	7.6	0.96	331.0		336.5
LDA1 f_{ck} =40 MPa	368.8	9.4	0.86	308.7		336.5
LDA1 f_{ck} =32 MPa	339.6	7.8	0.89	/	288.5	313.5
LDA2 f_{ck} =40 MPa	372.6	7.1	0.98	335.0		336.5
G1 f_{ck} =32 MPa	375,7	13.1	0.95	333.1	350.5	314.5
GR1 f_{ck} =32 MPa	384,0	12.9	0.95	338.45	360.1	314.5

$F_{stat1,Rk}$ Annex B – Eurocode 4
$F_{stat2,Rk}$ Annex D – Eurocode 0
F_{av} –average shear resistance of tested specimens
δ_{av}– longitudinal slip that correspond to the average max. load

Average force-slip diagrams of each tested group are shown in Figure 9. Two reference values $F_{Rk1}$and F_{Rk2} for the diagrams presented in Figure 9, represent a characteristic design value of shear resistance calculated according to Eurocode 4 with actual characteristics of the materials used in experiments $F_{Rk,exp}$.

The diagrams in Figure 9 clearly illustrate that the shear resistance of all the tested specimens is higher than the values of characteristic shear resistance ($F_{Rk,exp}$) given by [1] assuming no reduction due to a smaller space between studs.

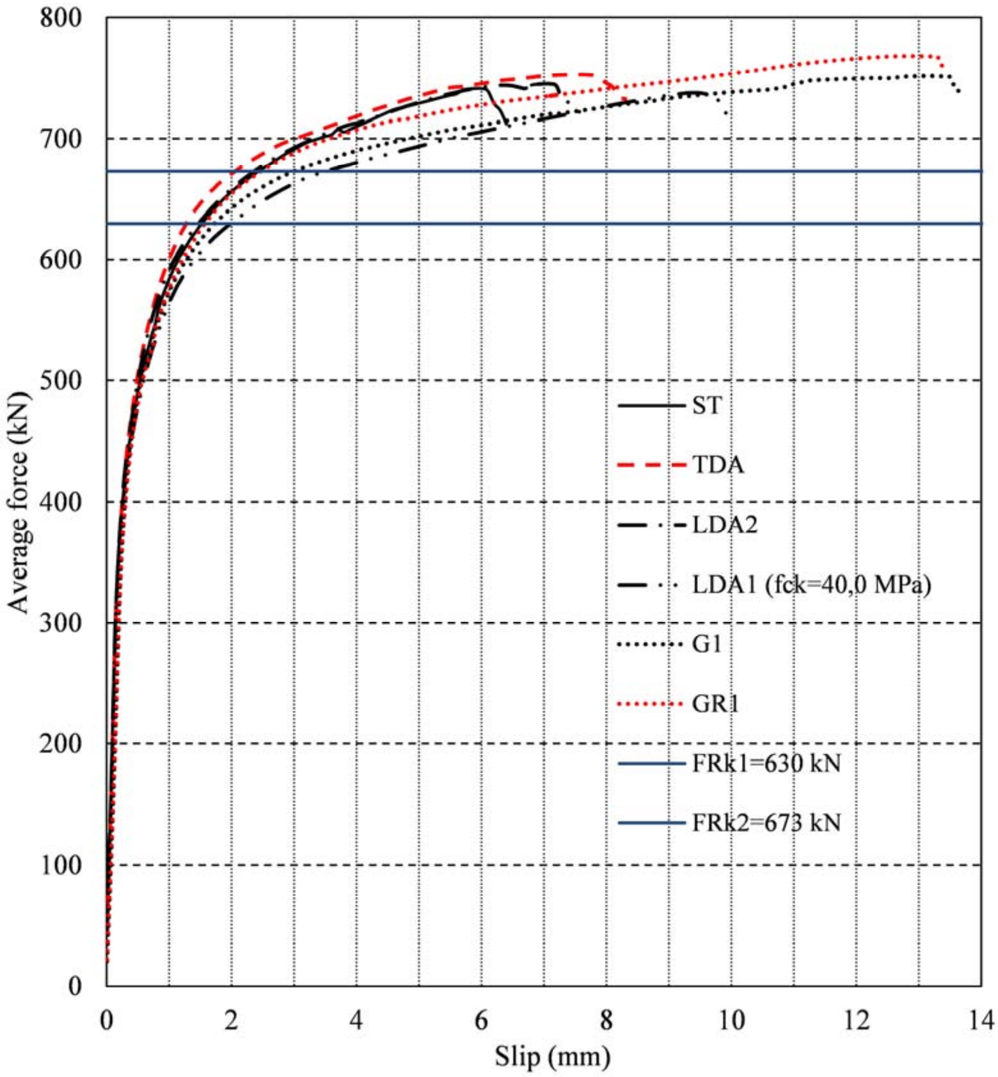


Figure 9. Load-slip Behavior for Test Specimens

4.1. Characteristics of Failure Mechanisms

The results of specimens ST, which comply with [1], create reference data for analysis of results and behavior of other specimen types. In case of specimens with group types ST, LDA1, LDA2 and TDA there was the shear failure of headed studs. The higher measured slip of the LDA1 type specimens, see Table 4, column (3), in respect to the ST specimens indicate that major plastic deformations (crushing) of concrete in the zone around the headed studs occurred.

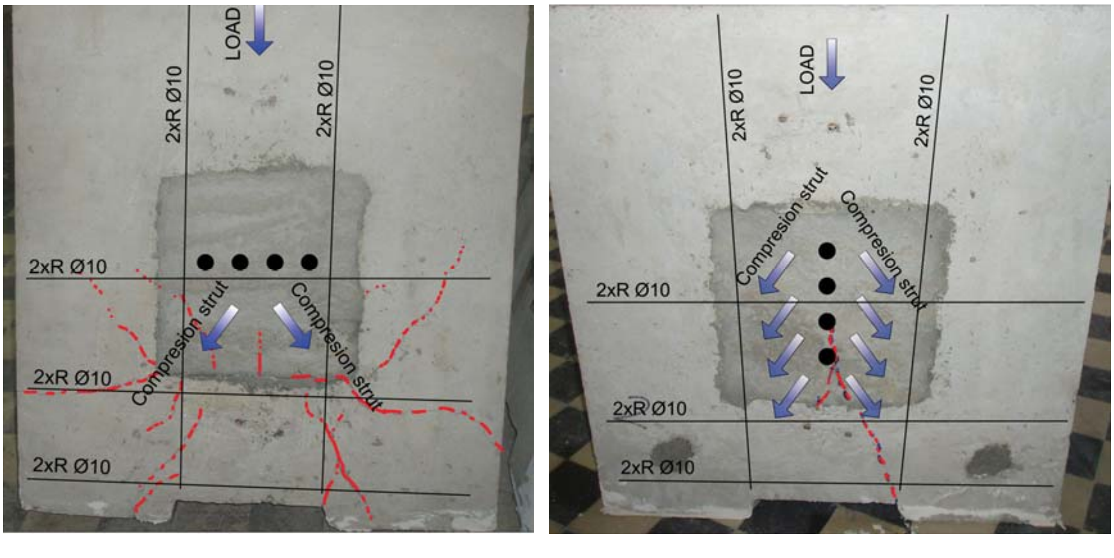
In series G1 and GR1 the failure also occurred by shearing of the headed stud. However, in the case of G1 and GR1 series, considerably higher deformations at failure were measured, which was affected by application of lower strength of infill concrete (f_{ck}=32MPa) in respect to the infill concrete used for the first phase specimens. Higher deformations of G1 and GR1 specimens are also resulted by higher plastic deformations of concrete close to headed studs group, which gave rise to combined deformations of headed studs due to bending and tension.

The ST specimens failure, at the average value of ultimate force of 762 kN, occur to be shear of studs with the average ultimate deformation of 6.6 mm. No cracks occurred in the reinforced concrete slab at failure. In the case of TDA type specimens, failure also occurs by studs shear, and the value of shear resistance is 754 kN with the failure deformation of 7.75 mm. Considering that the TDA specimens were constructed in full accordance with [1], in terms of arrangement of headed studs, such behavior of the specimen is expected. However, the TDA type specimen, showed considerable cracks in reinforced concrete slab around studs and in the slab area in front of the studs (see Figure 10a). The cracks appeared in concrete slab as early as with loads of 595 kN to 620 kN. The onset of these cracks is caused by high tensile stresses, perpendicular to main compression stresses. The force flow (strut-and-tie model) and the scheme of the cracks in the concrete slab are presented in Figure 10a. It is clear that the concentration of the load, in the section of the slab at the location of the headed stud row, initiates the formation of two heavily compressed diagonals. In this way the infill part of concrete acts as rigid element to precast concrete slab. Higher strength of infill concrete slab compared to precast slab results in larger cracks of the precast slab. Confinement effects are neglected because the precast slab concrete has similar properties as the infill concrete. No separation between infill part of concrete and precast concrete slab was noticed.

Analyzing the presented mechanism of the load transfer, it is clear that by adequate layout of reinforcement in the slab, cracks could be reduced or even fully prevented.

Table 4. Comparison of Experimental Results

Specimens	$F_{av}/F_{ST,av}$	$\delta_{av}/\delta_{ST,av}$	$\frac{F_{stat1,Rk}}{F_{ST,stat1,Rk}}$	$\frac{F_{stat1,Rk}}{F_{Rk,exp}}$	$\frac{F_{stat2,Rk}}{F_{Rk,exp}}$
(1)	(2)	(3)	(4)	(5)	(6)
ST	1.000	1.00	1.00	0.972	/
TDA	1.016	1.25	1.01	0.983	/
LDA1 f_{ck}=40 MPa	0.995	1.56	0.944	0.917	/
LDA1 f_{ck} =32 MPa	0.916	1.30	/	/	0.92
LDA2	1.005	1.18	1.024	0.995	/
G1	1.012	2.14	1.018	1.057	1.113
GR1	1.036	2.11	1.034	1.074	1.143



a) TDA b) LDA1

Figure 10. Cracks in the Concrete and a Model of Internal Forces

Obtained values of shear resistance of the G1 and GR1, LDA2 types specimens, with two headed studs in the direction of the load at a smaller distance than recommended 5d are the same or higher than the obtained ultimate values of shear resistance of standard ST type specimens (see Table 4). Due to the small distance, the headed studs in the group have approximately the same deformation, which results in the uniform redistribution of forces between the headed studs. The common work of all the studs in the group and redistribution of load between the individual headed studs resulted in smaller differences between the measured failure load values between the specimens of the same type. Hegger et al. [5] used concretes with higher strengths and two studs in a row (h_{sc}/d =80/19 mm=4.21) with the distance of 2d in the direction of the force, and obtained only 5.4% lower shear resistance. The average value of the shear resistance of tested specimens of the G1, GR1 and LDA2 types are within the range of ±3% in respect to the standard ST specimen. The statistic values of the shear resistance ($F_{stat1,Rk}$, $F_{stat2,Rk}$) of the specimens of these groups of headed studs have higher values than the shear resistance of the standard specimen. It should be pointed out that G1, GR1 and two LDA1 specimens were monolitized with the concrete having strength of f_{ck}=32 MPa. Comparison results of tested specimens with headed studs in line of force with results in [5] indicate the importance of headed studs height to diameter ratio (h_{sc}/d).

The average value of failure force of LDA1 type specimens (f_{ck}=40MPa) of 757kN is 0.6% smaller than the average value obtained for standard specimens (ST). Failure forces of LDA1 specimens are 2-19% higher than the characteristic values. Shear resistance of individual LDA1 type specimens were characterized by slightly higher deviation from the average value for this type. The characteristic shear resistance of LDA1 is 8.0% smaller than the characteristic shear resistance of ST specimens.

It should be mentioned that at failure of LDA1 type specimens the cracks in the concrete slab did not emerge, or were very small, and they emerged only at the loads very close to the shear resistance of the specimen. In case of longitudinal orientation of headed studs, when the load is transferred a larger number of compressed diagonals in concrete is formed, as opposed to the transversal orientation of studs (TDA type) in which case two compressed diagonals are formed, see Figure 10. The flow of internal forces (strut-and-tie model) in the concrete slab presented in the Figure 10b clearly illustrate that in the case of LDA1 headed stud group, two horizontal rows are engaged for reception of tensile forces.

However, one must take into account both the way of application of load by the headed studs group onto the concrete slab, and the internal flow of forces, and provide adequate reinforcement in the concrete slab. Also, the appropriate reinforcement for transfer of longitudinal shear force in concrete must be present.

G1 and GR1 groups are identical in geometrical terms, but they were constructed with the different type of concrete slabs. Combined type of failure is characteristic for both types of specimens. Average values of ultimate failure load of 747kN for the G1 type specimens, i.e. 753kN for GR1 type specimens demonstrate that transversal reinforcement in the slab does not have considerable influence on the shear resistance of the headed stud group. The same conclusion was presented in Okada's paper [2]. The slip at the failure of G1 specimen was 11.8 mm, while for GR1 specimen it was 12.8 mm. From the presented results and load-slip characteristics of G1 and GR1 specimens, the conclusion can be drawn that the reinforcement of the slab does not affect the shear resistance and behavior of the group of headed studs. The role of reinforcement in the slab is primary important because of the flow of force inside the reinforced concrete slab. The reinforcement is important for strut-and-tie model in concrete slab in the zone with considerable concentrated load onto the concrete slab, at the location of the headed stud group.

The strength of the infill concrete plays a dominant role. The higher shear resistance and stiffness of the shear connector is achieved using the high strength infill concrete. Therefore, it is cost-effective to use it.

5. SHEAR RESISTANCE OF STUDS GROUP

Shear resistance of headed studs in a group may be predicted as a sum of the shear resistance of individual headed studs. In this approach, it is important to take into account a position of the shear stud in a group. A similar interpretation of the shear resistance of two headed studs is provided by Hegger, Sedlacek et al. [5].

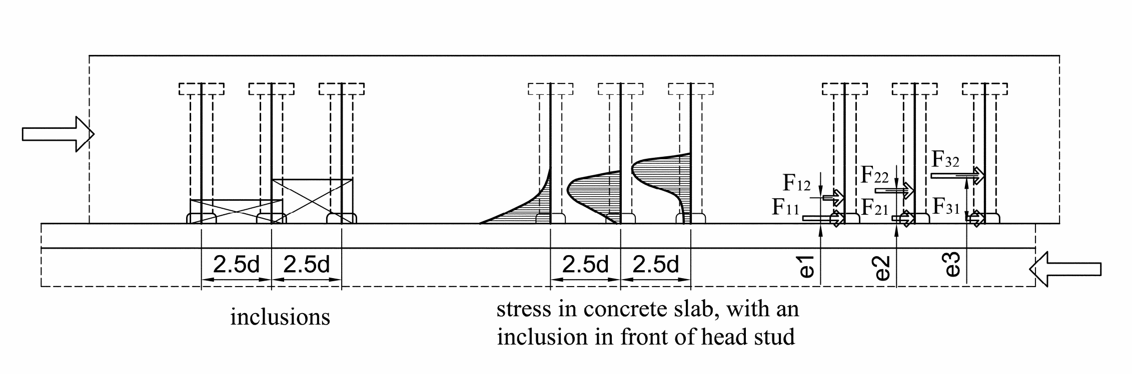


Figure 11. Load Distribution Model for the Group of Headed Studs

The first stud in the column can achieve the full shear resistance. The shear failure occurred in this stud. It can be considered that the shear resistance of concrete has been used in a layer which is approximately equal to the diameter of the headed stud, at the first stud. The second headed stud in the row behaves as a headed stud with an inclusion in front of it, as shown in Figure 11. It is similar to the case of a stud installed in the corrugated steel sheet. With the same analogy the shear resistance of the subsequent studs in the row can be analyzed. The second, third and other studs may have the same or slightly lower shear resistance which directly depends on the height of the shank of headed stud at which the concrete pressure ocures. These headed studs are characterized by the shear-bending failure mode. According to Figure 11 it can be define as follow: $F_{11}>F_{21}>F_{31}$, $F_{12}<F_{22}<F_{32}$ and $e_1<e_2<e_3$. The higher force F_{2i} and eccentricity e_i results in higher bending moment in the shank. The higher values of bending moment results in a dominate bending-shear failure which is typical for the third and the fourth head stud in the row. Forces F_{ij} are influenced by stud diameter, clearance between studs and the bending stiffness of headed stud. The concrete strength is one of most dominated factors to shear resistance of headed studs group. Thickness of concrete layer that will be engaged for load transfer depends not only on geometry but also the concrete class. In a case of single headed studs, the concrete class determines the form of the stud failure, Lam et.al. [11] and Oehlers [6].

Okada et al. in [2] analyzed the influence of the stud distance to the group shear resistance in the similar way. They suggested the reduction factor for the group of studs which is in relation to concrete strength and longitudinal spacing factor $C_1=d_l/d$ (d_l - longitudinal distance between studs).

According to [2] for specimens LDA1,G1 and GR1 following values could be calculated:

- concrete C30/37

$F_{Rk}= \eta \cdot F_{Rk,exp}=(0.021 \cdot C_1 + 0.73)\, F_{Rk,exp}$ (4)

for d_1=45mm → $C_1=d_1/d$=2.8125 → $\eta=0.021 \cdot C_1 + 0.73=0.789$

for d_1=50mm → $C_1=d_1/d$=3.125 → $\eta=0.021 \cdot C_1 + 0.73=0.796$

- concrete C40/50

$F_{Rk}= \eta \cdot F_{Rk,exp}=(0.016 \cdot C1 + 0.80)\, F_{Rk,exp}$ (5)

for d_1=45mm $C_1=d_1/d$=2.8125 → $\eta=0.016 \cdot C_1 + 0.80=0.845$

for d_1=50mm $C_1=d_1/d$=3.125 → $\eta=0.016 \cdot C_1 + 0.80=0.85$

Table 5 and Figure 12 present the test result and Okada's recommendation for the group shear resistance using the empirical reduction factor η. The best agreement is obtained for LDA1 specimens. Okada analyzed specimens with three studs in force direction. Therefore, specimens LDA1 with four studs have the good match with the estimated resistance. This may indicate that the diameter and the total number of studs are also parameters which are important for shear resistance of studs group. Shear resistance of tested specimen ST should not be reduced. There are significant deviations between results for G1 and GR1 series and reduced resistance according to [2]. This deviation indicates that the number of studs in the force direction and the total number of studs in the group are relevant factors for the shear resistance. It is necessary to take these variables into account for the reduction parameter η.

Table 5. Comparisons of the Specimens Shear Resistance with the Estimation based on [2]

Specimens	$F_{Rk,exp}$ acc. to EC4	$F_{stat1,Rk}$	$F_{stat2,Rk}$	Distance between studs	Acc. to Okada [2]
	[kN]	[kN]	[kN]	[mm]	[kN]
(1)	(2)	(3)	(4)	(5)	(6)
ST f_{ck}=40 MPa	336.5	327.0		100 mm 6.25d	302.5
LDA1 f_{ck} =40 MPa	336.5	308.7		50 mm 3.125d	286.0
LDA1 f_{ck} =32 MPa	314.0		288.5	50 mm 3.125d	250.0
G1 f_{ck} =32 MPa	313.5	331.1	350.5	45 mm 2.8d	247.5
GR1 f_{ck} =32 MPa	315.0	338.4	360.0	45 mm 2.8d	248.0

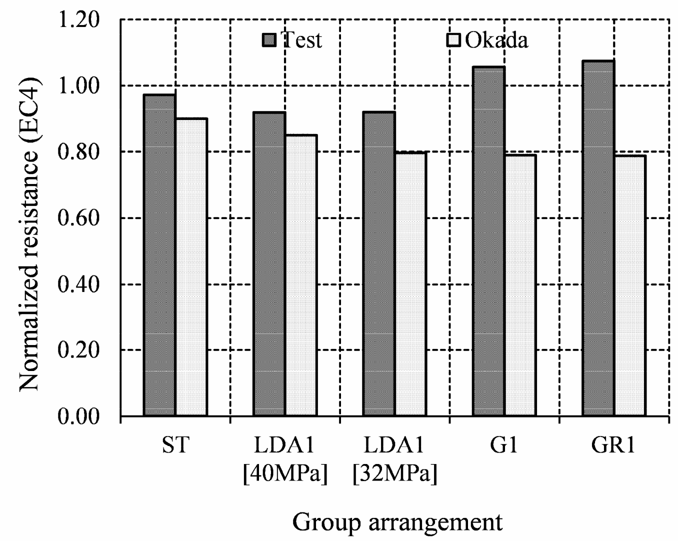


Figure 12. Test Results Comparison with EC4 and Okada

If higher concrete classes are used, the layer of concrete engaged for load transfer is lower, and the loading of the headed studs is concentrated closer to the root. This results in smaller bending of the headed studs. Use of higher concrete classes comprises that the layers of concrete engaged for load transfer on the second, third and other headed studs also lie closer to the root of the headed stud. In Figure 13 are displayed deformed headed studs after the failure of LDA1 specimen. One may observe from the Figure 13 that the fourth headed stud in the column has broken. It should be noted that the third headed stud endured considerably greater deformation than the first two studs. This justifies the assumption that the last stud is exposed to the highest bending moment due to larger lever arm of the resulting stresses in the concrete.

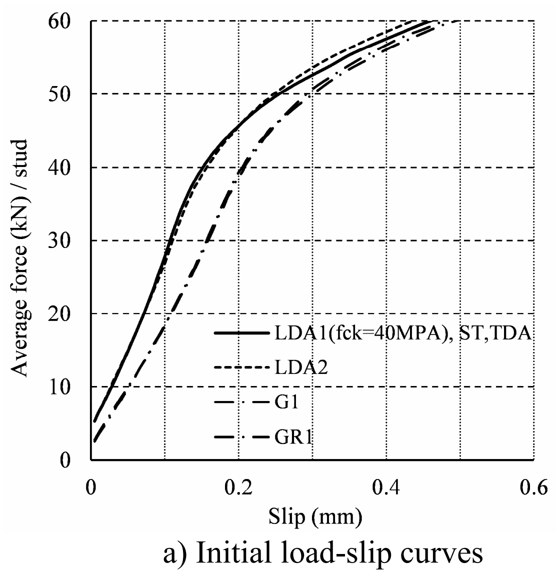
Figure 13. Deformed Shape of Headed Studs after Failure of LDA1 Specimen

6. DEFORMATIONS AND STIFFNESS

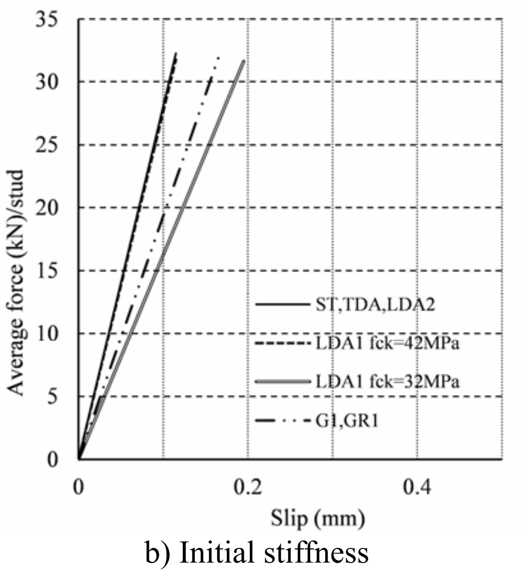
Stiffness and deformability of the shear connectors is an important input for the analysis of composite beams. Beam deflections are affected by the slip in the shear connection interface. These effects are not covered in design codes for composite beams with the full shear connection and the standard arrangement of studs. Since grouped arrangement of headed stud is a non-standard arrangement, the slip in shear connection was analyzed. The slip of the shear connection in the group of studs is also compered to the standard arrangement for the serviceability load level (SLS). According to values in Table 2 column (8), the slip of the shear connection with four headed studs in the group arrangement is equal to the slip for the standard arrangement. Consequently, it may be expected that the deflection of a composite beam with grouped studs will be equal to the deflection of a beam with the standard studs arrangement for SLS.

Characteristics of the concrete, used for the monolitization, have a dominant influence on the stiffness of the shear connection. It is well known, that the higher the concrete class is the lower plastic deformation of the concrete is and thus the deformation of the headed studs. This would result in a higher stiffness of the shear connection. Transfer of the load from the headed studs into the concrete slab at initial values of the load occurs through the concrete layer near the root of the headed stud. Application of the low strength concrete comprises onset of plastic deformations and cracks in the layer of concrete near the root of the headed stud, even at low values of the force. It further results in bending of headed studs and engaging of a thicker concrete layer for load transfer. Early plastic deformations and cracks in the concrete slab in case of the low strength concrete comprise higher initial deformations of the shear connector and higher ultimate deformations at failure.

In Figure 14 the initial parts of load-slip curves are presented calculated for one headed stud.



a) Initial load-slip curves



b) Initial stiffness

Figure 14. Initial Stiffness for Tested Groups

It can be observed in Figure 14b that the tangential modules of ST, TDA, LDA1 and LDA2 type specimens are almost identical. The tangent modules coincide for specimens G1 and GR1. It can be concluded that the arrangement of headed studs in the group and the orientation of the group do not affect the initial stiffness of the group importantly, but it primarily depends on the strength of concrete used for monolitization. It was noted that the difference in slip of the various types of studs groups for the load level of 0,7 P_{Rk} (see Table 2 and Figure 15) is within the range of 0.1 mm.

Same slip deformations of the G1 and GR1 group types demonstrate that transversal reinforcement has no important influence on the group.

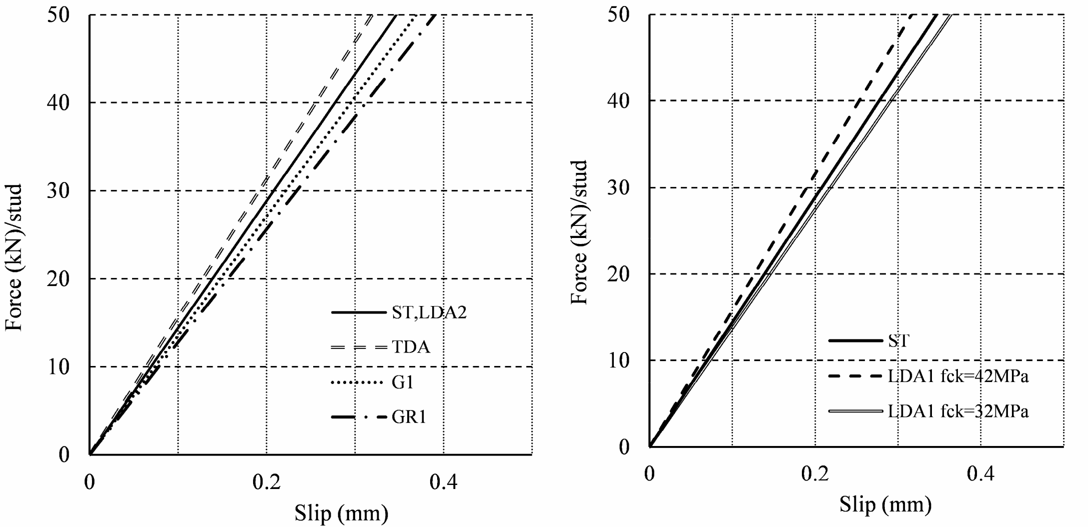


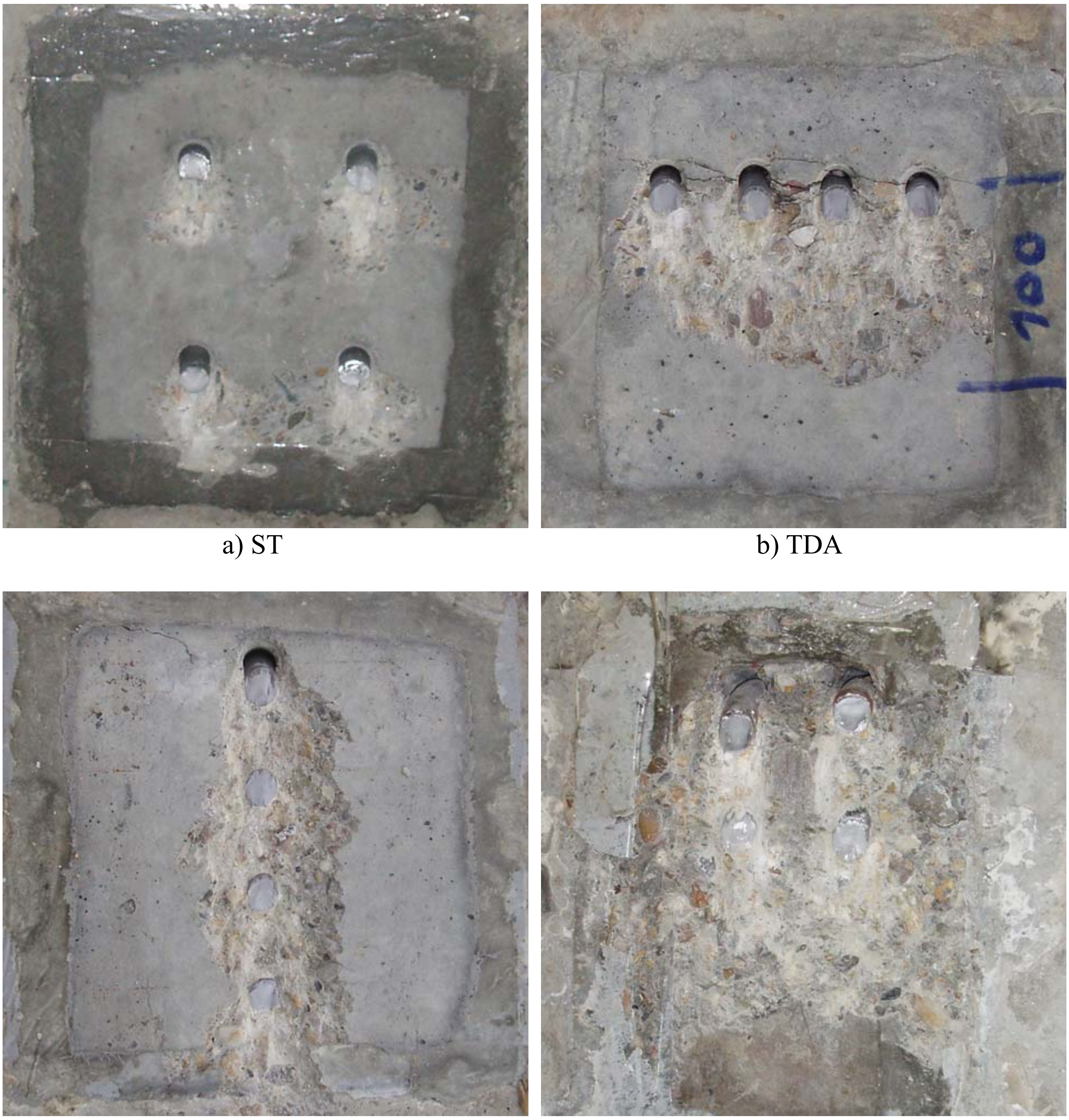
Figure 15. Stiffness k_{sc} for Tested Groups of Headed Studs

Table 6 presents the values of separation between steel beam and concrete slab. Average specimen values of separation for the maximum values of shear force are presented. Presented values are uniform for all type of specimens. Grouped arrangement of headed studs has no influence on separation between steel beam and concrete slab.

Table 6. Separation between Steel Beam and Slab

specimen	ST	TDA	LDA1	LDA2	G1	GR1
	Average value - separation between the beam and the slab [mm]					
a	1.06	1.12	0.81	0.53	1.0	0.85
b	1.25	0.67	1.12	0.47	0.87	0.66
c	1.09	1.03	0.90	0.81	1.17	-
d			1.03		1.16	0.84
e			1.2			

Analyzing the damaged areas in concrete around headed studs (Figure 16) it may be observed that the zones of plastic deformations in concrete are different both in form and the volume. Even though the TDA specimens comply with [1], the arrangement of studs in the slab and the application of force onto the slab as well as the internal force flow (see Figure 10) result in a considerably larger zone of plastic deformations in the concrete slab. It was expected that the arrangement of headed studs in LDA1 specimen will undergo significant plastic deformations in the concrete slab, especially when it is considered that the space between the headed studs is reduced, but this was not experimentally proved. This indicates that the internal flow of forces in the concrete slab has a considerable effect on the resistance of grouped shear connectors.



a) ST b) TDA

c) LDA1 d) G1 and GR1

Figure 16. Crashing Zone (Failure Mode) in Concrete Slab

7. CONCLUSIONS

The experimental study demonstrated that headed studs groups can be efficiently used when welded in the group even if the distance between studs is smaller than the minimum required ($5d$) [1]. The findings of the study are given below:

- Shear resistance of a group of four headed shear studs Ø16 mm, in a row in the longitudinal direction at the distance of $3d$, is equal to 92% of the sum of the shear resistance of individual shear connectors in the solid concrete slab.
- Two headed studs in a longitudinal direction at the distance of $2.5d$ with the ratio of the height and diameter of a stud bigger than $h_{sc}/d \geq 6.25$ have the same shear resistance as two individual studs, when the concrete strength $f_{ck} \geq 32$ MPa is used.

- Recommendation on the reduction factor given in Okada et all [2] may be used as the safe prediction of the shear resistance of the specimen with four studs in the row, the LDA1 specimen., Acc. to [2] the reduction factor would be 0.93 and 0.87 for the characteristic resistance of the specimen with the concrete strength f_{ck} = 40 MPa, and f_{ck}=32 MPa; respectively. Further studies are on-going to derive an accurate prediction for more than three headed shear studs in the group.
- The stiffness and deformability of the connection depends of the concrete strength. Stiffness of studs group in specimen with concrete strength f_{ck} = 32 MPa is 10% lower than the stiffness of the same group of studs in specimen with concrete strength f_{ck} = 40 MPa.
- A group of headed studs (two or four in the row) with reduced distance, less than 5*d*, may be used instead of a larger diameter single headed stud. The group has ductile behavior at the longitudinal slip greater then 6,0 mm;

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NOTATION

d	diameter of the shank of a stud connector
h_{sc}	overall nominal height of a stud connector
ST	standard test
LDA	longitudinal distance arrangement
TDA	transversal distance arrangement
G	group arrangement without reinforcement near the group of headed studs
GR	group arrangement with reinforcement near the group of headed studs
F_u	the shear resistance of tested specimens
$F_{u,min}$	minimum shear resistance of tested specimens with a same group of headed studs
$F_{u,max}$	maximum shear resistance of tested specimens with a same group of headed studs
F_{av}	average shear resistance of tested specimens with a same group of headed studs
δ_{max}	maximal longitudinal slip at failure load
δ_{av}	longitudinal slip that correspond to the average max. load
k_{sc}	stiffness of shear connection and Stiffness of a shear connector
$F_{Rk,exp}$	characteristic shear resistance on the basis of experimental characteristics of the material
$f_{c,cube}$	characteristic compressive cube strength of concrete at 28 days
f_{ck}	characteristic compressive cylinder strength of concrete at 28 days
f_u	the ultimate tensile strength
E_{cm}	secant modulus of elasticity of concrete
P_{Rk}	characteristic value of the shear resistance of a single connector
α	coefficient
$F_{stat1,Rk}$	characteristic resistance according to EC4 Annex B
$F_{stat2,Rk}$	characteristic resistance according to EC0 Annex D
m_x	mean of the n sample results
k_n	coefficient for the 5% characteristic value
V_x	coefficient of variation
s_x	estimate value of the standard deviation

η the reduction factor [2]
n number of same specimens
C_1 longitudinal spacing factor [2]
d_l longitudinal distance between the studs [2]

REFERENCES

[1] EN1994-1-1, Eurocode 4, "Design of Composite Steel and Concrete Structures. General Rules end Rules for Buildings", CEN (European Committee for Standardization), December 2004.

[2] Okada, J., Yoda, T. and Lebet, J.P., "The Study of the Grouped Arrangements of Stud Connectors on Shear Strength Behavior", Structural Eng./Earthquake Eng.,JSCE, Vol. 23, No. 1, pp. 75-89, April 2006.

[3] EN1994-2, Eurocode 4, "Design of Composite Steel and Concrete Structures. General Rules and Rules for Bridges", CEN (European Committee for Standardization), 2005.

[4] Shim, C.S., Lee, P.G., Kim, D.W. and Chung, C.H., "Effects of Group Arrangement on the Ultimate Strength of Stud Shear Connection", Proceedings of the 2008 Composite Construction in Steel and Concrete Conference VI, ASCE Conf. Proc. doi:10.1061/41142(396)8.

[5] Hegger, J., Sedlacek, G., Doinghaus, P. and Trumpf, H., "Studies on the Ductility of Shear Connectors when using High-strength Steel and High-strength Concrete", Conf. Proc. Ed. by Eligehausen, R." Connections between Steel and Concrete", Stuttgart, Germany, 10-12 September 2001, pp. 1025-1046.

[6] Oehlers, D.J., "Stud Shear Connectors for Composite Beams", PhD Thesis, The University of Warwick, 1980, /http://go.warwick.ac.uk/wrap/3977.

[7] Pallarés, L. and Hajjar, J.F., "Headed Steel Stud Anchors in Composite Structures. Part I Shear", Journal of Constructional Steel Research, 2010, Vol. 66, pp. 198-212.

[8] BS 5950-3.1:1990, "Structural Use of Steelwork in Building. Part 3 Design in Composite Construction", UK British Standard Institution, 1990.

[9] EN1992-1-1, Eurocode 2, "Design of Concrete Structures, General Rules and Rules for Buildings", CEN (European Committee for Standardization), 2004.

[10] EN1990, Eurocode 0, "Eurocode - Basis of Structural Design", CEN (European Committee for Standardization), 2002.

[11] Lam, D., Asce, M. and El-Lobody, E., "Behavior of Headed Stud Shear Connectors in Composite Beam", Journal of Structural Engineering, Vol. 131, No.1, ASCE, January 2005.

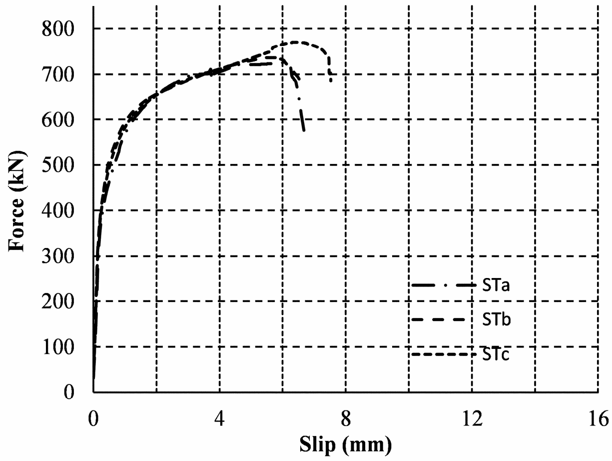
[12] Ollgaard, J.G., Slutter, R.G. and Fisher, J.W., "Shear Strength of Stud Connectors in Lightweight and Normal-weight Concrete", Engineering Journal, AISC 1971, Vol. 8, No. 2, pp. 55-64.

[13] Oehlers, D.J. and Bradford, M.A., "Elementary Behaviour of Composite Steel and Concrete Structural Members", Butterworth-Heinemann, 1999.

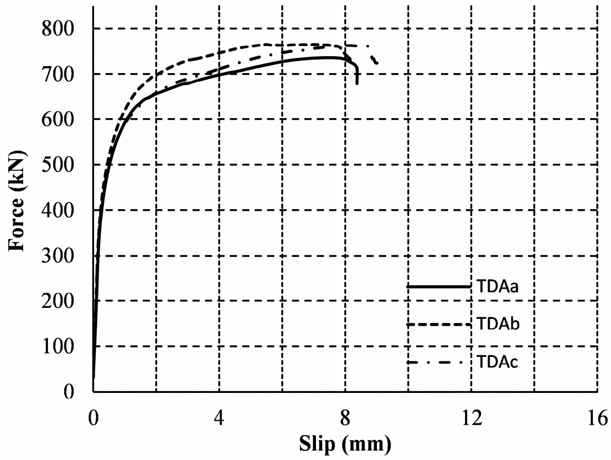
[14] Spremic, M., Fric, N., Markovic, Z. and Budjevac, D., "Shear Capacity of Anchor Plate with Group of Headed Studs", International Conference, GNP 2010, Zabljak, 15-19 February 2010, pp. 649-655.

APENDIX 1

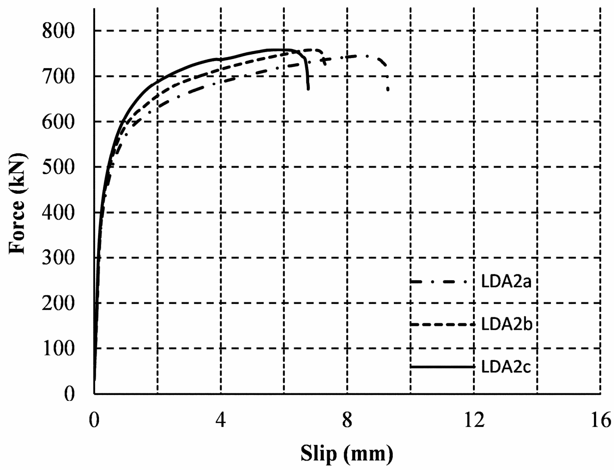
Test results of all tested specimens



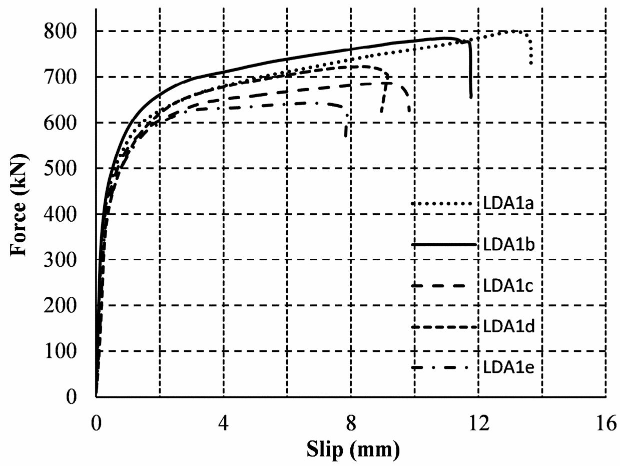
a) Specimens ST



b) Specimens TDA



c) Specimens LDA2



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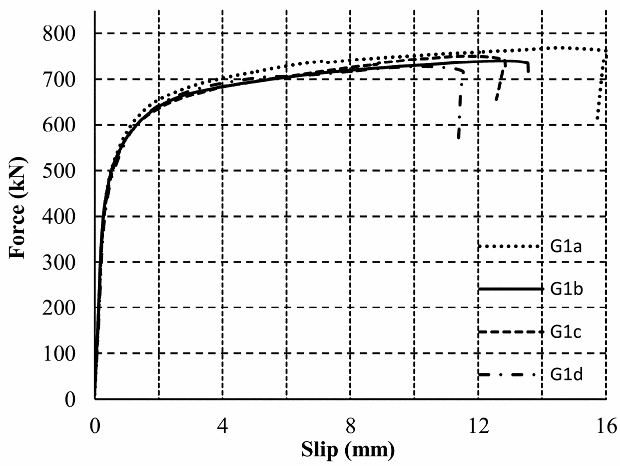
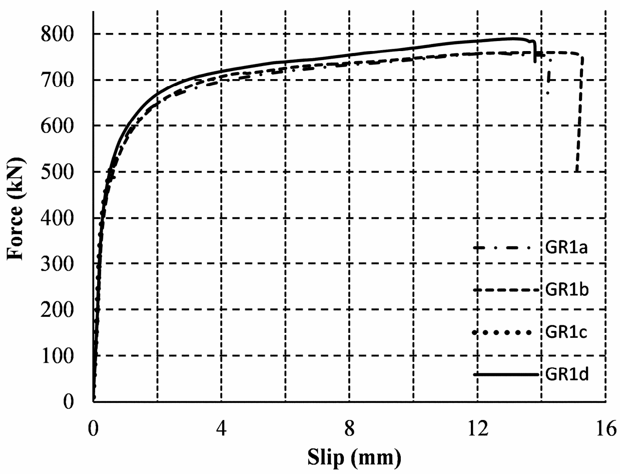


e) Specimens G1



f) Specimens GR1