Steel and Composite Structures, *Vol. 26, No. 3 (2018) 375-386*
DOI: https://doi.org/10.12989/scs.2018.26.3.375

FE validation of the equivalent diameter calculation model for grouped headed studs

Milan Spremic [*1], Marko Pavlovic [2a], Zlatko Markovic [1b], Milan Veljkovic [2c] and Dragan Budjevac [1d]

[1] *University of Belgrade, Faculty of Civil Engineering, Bulevar kralja Aleksandra 73,11000 Belgrade, Serbia*
[2] *Delft University of Technology, Faculty of Civil Engineering and Geosciences, Stevinweg 1,2600 GA Delft, The Netherlands*

(*Received September 5, 2017, Revised October 30, 2017, Accepted November 3, 2017*)

Abstract. Existing design codes for steel-concrete composite structures give only general information about the shear connection provided by headed studs in group arrangement. Grouting of the openings in prefabricated concrete slabs, where the grouped headed studs are placed in the deck pockets is alternative to cast-in-place decks to accomplish fast execution of composite structures. This paper considers the possibility to reduce the distance between the studs within the group, bellow the Eurocode limitations. This may lead to increased competitiveness of the prefabricated construction because more studs are placed in the group if negative effectives of smaller distances between studs are limited. The main purpose of this work is to investigate these limits and propose an analytical calculation model for prediction of the shear resistance of grouped stud arrangements in the deck pockets. An advanced FEA model, validated by results of push-out experiments, is used to analyze the shear behavior of the grouped stud with smaller distance between them than recommended by EN 1994-1. Calculation model for shear resistance, which is consistent with the existing Eurocode rules, is proposed based on a newly introduced equivalent diameter of the stud group, d_G. The new calculation model is validated by comparison to the results of FE parametric study. The distance between the studs in the longitudinal direction and the number of stud rows and columns in the group are considered as the main variables.

Keywords: group of headed studs; equivalent diameter; shear connection; finite element method

1. Introduction

Headed studs in group arrangement are a type of discontinuous shear connection in prefabricated composite beams. Grouped arrangement of headed studs is a very good alternative for rigid block shear connectors, because they are providing more ductility and are easier to execute. Such shear connection was originally used in prefabricated composite bridges. However, the dimensions of the group of the studs are often larger than the block shear connectors having equivalent shear resistance. In order to have dimensions of the group of headed studs similar to the dimensions of a block shear connector, it is necessary to reduce the distance between the studs in a group. With reduced dimensions of the group, the precast concrete slab can be constructed with the smaller openings where the studs are fitted and grouted during the execution. The current design codes and recommendations do not provide sufficient information on how to determine the shear resistance of the group of headed studs when distances are smaller than the limitations provided for individual, non-grouped, headed studs. The longitudinal shear connection provided by the groups of studs can be characterized as a discontinuous shear connection. The EN 1994-2 (2008) allows the use of headed studs in group arrangement in composite bridges. However, no calculation model for shear resistance in case of very closely spaced studs is given. The load bearing capacity of the group of headed studs was analyzed by Okada *et al*. (2006) and Shim *et al*. (2008). Both studies investigated the shear resistance of headed studs in group where the distance between the studs in the direction of the shear force (the longitudinal direction) was reduced with respect to the limitations given in the design codes, $5d$ and $6d$ according to EN 1994-1-1 (2004) and AISC (2005), respectively. Okada *et al*. (2006) and Shim *et al*. (2008), proposed equations for calculation of shear resistance of a group of headed studs. In both cases, the reduction of shear resistance due to group behavior is defined as a function of distance between the studs in the longitudinal direction. Okada *et al*. (2006) calculation model additionally incorporates the influence of characteristic compressive strength of concrete.

Push-out tests of headed studs in group arrangement with reduced distance between studs were realized in two research studies, Shim *et al*. (2008) and Spremic *et al*. (2013).

It is shown in Spremic *et al*. (2013) and Spremic *et al*. (2017b) that the shear resistance of a group of headed studs is a function of many geometrical parameters of the group. Parametric study performed by using Finite Element (FE)

*Corresponding author, Ph.D., Assistant Professor,
E-mail: spremic@grf.bg.ac.rs
[a] Ph.D., Assistant Professor
[b] Professor
[c] Professor
[d] Professor

http://www.techno-press.org/?journal=scs&subpage=6 ISSN: 1229-9367 (Print), 1598-6233 (Online)

modeling which is validated by experiment results is presented in this paper. The results show more in-depth influence of: number of studs in a group, layout of studs in a group, headed studs' height, and distance between studs in the longitudinal direction.

2. Previous experimental results

Most of existing push-out tests of headed studs in group arrangement are realized with a group of nine headed studs in the 3×3 layout, see Fig. 1. These groups of nine headed studs are applicable for shear connection in prefabricated bridge structures. The layout and results of previous experimental research studies are presented in Table 1. The number of studs in force direction and perpendicular to the force direction are defined by variables n_r and n_c respectively. The distance between the adjacent studs in the group is defined with non-dimensional values $\varepsilon_l = e_l/d$ and $\varepsilon_t = e_t/d$. Range of geometrical characteristics shown in Fig. 1 used in push-out experiments are given in Table 1. The overall height of stud after the welding was marked as h_{sc}. These symbols will be used further in the paper.

Okada *et al.* (2006) and Shim *et al.* (2008) investigated the influence of additional reinforcement in a hole of precast concrete slab with grouped studs. The presented results show that there is no influence of reinforcement to shear resistance, see Spremic *et al.* (2013), An and Cederwall (1996), and Okada and Lebet (2000).

Eurocode 4 (2004) defines the shear resistance of studs as a function of stud and concrete material characteristics, and the geometry. It is easy to show that the value of shear resistance due to shear failure and concrete failure is equal for concrete compressive strength 35.0 MPa and headed stud's height larger than 4*d*. It has been shown by push-out tests in Pallarés and Hajjar (2010), that the shear failure of studs is the governing failure mode if headed studs have height 4*d* or larger.

Eurocode 4 (2004) allows the use of studs having height less than 4*d*, but prescribes the reduction of the shear resistance.

The specimen failure is governed by the shear failure of the studs if following requirements are satisfied:

- the distance between the headed studs is higher or equal to 5*d*,
- the concrete compressive strength is higher than 35.0 MPa, and
- the height of shear studs is higher or equal to 4*d*.

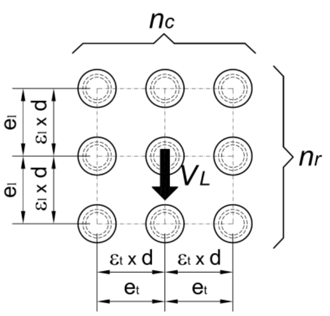


Fig. 1 Designation of geometry parameters of the group of headed studs

Table 1 Existing results of push-out tests on grouped headed studs

	f_{cm} [MPa]	d [mm]	h_{sc} [mm]	$n_c \times n_r$	ε_t	ε_l	$P_{u,stud}$ [kN]
Okada and Lebet (2000) $f_{u,stud}$ = 530 MPa (with reinforcement)							
SP3-1	49.5	22	150	3×3	3	5	208
SP3-2	44.3	22	150	3×3	3	5	192
SP3-3	49.5	22	150	3×3	3	5	201
Okada and Lebet (2000) (without reinforcement)							
SP4-1	49.5	22	150	3×3	3	5	205
SP4-2	44.3	22	150	3×3	3	5	190
SP4-3	49.5	22	150	3×3	3	5	208
Shim *et al.* (2008) $f_{u,stud}$ = 490 MPa (NS – specimens without reinforcement)							
G25NS	49.5	25	190	3×3	3	5	1.0 P_{Rk}
G25OS	49.5	25	190	3×3	3	5	1.0 P_{Rk}
G25IS	49.5	25	190	3×3	3	5	1.0 P_{Rk}
G25OS-1	49.5	25	190	3×3	3	4	0.75 P_{Rk}
G25NS-2	49.5	25	190	3×3	3	3	0.7 P_{Rk}
G25OS-2	49.5	25	190	3×3	3	3	0.7 P_{Rk}
G25OS-2	49.5	25	190	3×3	3	3	0.7 P_{Rk}
Guezouli *et al.* (2013)							
SP1	56	19	100	3×3	4	5	158
SP2	56	19	100	3×3	4	5	158
SP3	56	19	100	3×3	4	5	158
Xu *et al.* (2012) $f_{u,stud}$ = 480 MPa							
QT1	>50	13	80	3×3	4	4.6	63.4
QT2	>50	13	80	3×3	4	4.6	60.18
QT3	>50	13	80	3×3	4	4.6	54.92
Xue *et al.* (2012) $f_{u,stud}$ = 460-490 MPa							
MD1-1	50	22	200	3×3	4.5	4.5	186
MD1-2	50	22	200	3×3	4.5	4.5	186

The concrete in the prefabricated deck pockets can easily be made of the compressive strength higher than 35.0 MPa. Therefore, it might be possible to reduce distances between headed studs without causing concrete bearing failure.

The results of ultimate shear resistance per stud in group $P_{u,stud}$, obtained in push-out tests, are presented in Table 1. It is shown that no needs for reduction of the shear resistance exists of the distances between studs are according to EN 1994-1. Shear resistance of a stud group is equal to the sum of resistance for single studs in a group.

Only two experimental research studies of shear resistance were realized with groups which are constructed with the reduced distance between studs, see Shim (Shim *et al.* 2008) and Spremic *et al.* (2013). In their push-out tests, the groups of headed studs were constructed with spacing between studs 40% smaller than minimum allowed by Eurocode 4 (2008).

Table 2 Results of push-out tests

	f_{cm} [MPa]	d [mm]	h_{sc} [mm]	$n_c \times n_r$	ε_t	ε_l	$P_{u,stud}$ [kN]
Spremic *et al.* (2013) $f_{u,stud}$ = 520 MPa							
Group with reduced distance between studs							
GR1-A	39.3/43.5	16	100	2×2	2.8	2.8	94.7
GR1-B	39.3/43.5	16	100	2×2	2.8	2.8	95.0
GR1-C	39.3/43.5	16	100	2×2	2.8	2.8	94.0
GR1-D	39.3/43.5	16	100	2×2	2.8	2.8	98.7
Standard push-out tests							
ST a	44.4/44.2	16	100	2×2	5.0	>5.0	97.1
ST b	44.4/44.2	16	100	2×2	5.0	>5.0	92.2
ST c	44.4/48.6	16	100	2×2	5.0	>5.0	96.4

2.1 Push – out tests

The aim of push-out tests is to investigate consequences of reduced distance between headed studs. Standard push-out tests are realized according to Annex B of EN 1994-1-1. Geometry of slab and steel section, procedure for preparation of test and testing procedure are in accordance with Annex B, EN 1994-1-1 (2008).

Two layouts of groups are realized for setting up the tests: the standard test (ST) and the one with four studs in a row in the direction perpendicular to the force (TDA). Both layouts are according to the recommendation given in EN 1994-1-1 (2004). In ST specimens, the distance between studs in force direction is larger than minimum prescribed value 5d.

The second group of specimens (G1 & GR1) are made with the distance of 2.8d between the studs to check possible reduction of the shear resistance. Differences in the setup of push-out tests G1 and GR1 are in the layout of concrete slab reinforcement. In case of specimens GR1 the reinforcement bars are in front of the studs. Also, this reinforcement is used to connect concrete slab with infill concrete in openings. Concrete slab used for specimen's type G1 has no the reinforcement in openings in front of studs group. Adopted reinforcement in concrete slab, defined by Annex B, EN 1994-1-1 (2008), was sufficient to prevent longitudinal shear failure of slab. Transverse reinforcement need to be calculated according to EN 1992-1-1 (2004) for shear connection with the larger value of shear load. The distance between the studs of 2.8d in the group G1 & GR1 is very close to the technological minimum of 2.2d. The connection with reduced distance between the studs in the group (G1 & GR1) has the same value of shear resistance as the connection with standard arrangement of studs (ST). Equal ultimate shear resistance of specimens G1 and GR1 confirmed the negligible influence of reinforcement, Spremic *et al*. (2013).

Push-out tests with four studs in a column, in force direction, LDA (longitudinal distance arrangement) were also realized. LDA specimens with four studs in a column and the distance between the studs of 3.7d, were used for definition of internal forces and stresses in connections.

GR1

(a)

G1

(b)

Fig. 2 Concrete slab longitudinally sliced through the middle of the group of studs (G1&GR1)

This layout of a group has no practical application like the group GR1, but it is important for understanding of internal forces in a connection.

Results of push-out tests for specimens ST and GR1 are presented in Table 2. The whole results of experimental research were presented in Spremic *et al*. (2013). The main conclusion based on the push-out tests is that it is possible to reduce distance between the studs and get the shear connection with full resistance. Slip of connection GR1, with reduced distance between the studs, with value of 14.0 mm, is 100% greater compared with the value of slip in connections ST. The crack in RC slab, see Fig. 2 is characteristic of pry-out failure (concrete failure) of shear connection. Failure of G1 and GR1 specimens was the shear failure of studs at the ultimate load. These facts indicate the combined failure mode of G1 & GR1 connection. This type of connection failure is characteristic for shear connection with short headed stud. Parametric study presented in this paper explains the influence of studs' height to shear resistance.

3. FE Model

Simulation of the push-out experiments, referred to Spremic *et al*. (2013), is performed using the ABAQUS-Explicit dynamic solver for quasi-static analysis by implementing non-uniform mass scaling. Fig. 3 shows one

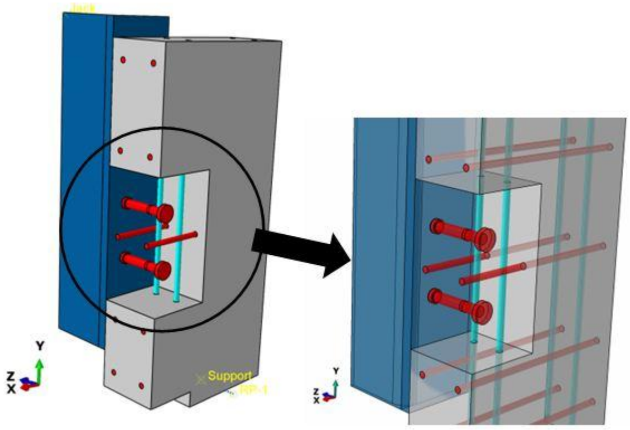


Fig. 3 FE model of push-out test – case ST

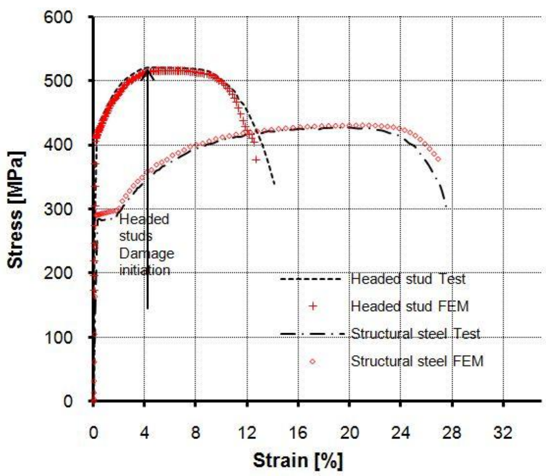


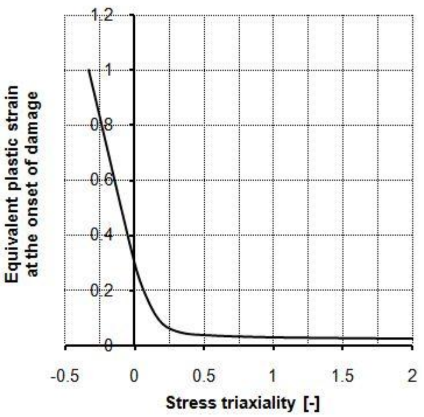
Fig. 4 Stress – Strain curves FEA vs. Tensile coupon tests

quarter of the push-out specimen which was modeled using two symmetry boundary conditions in order to shorten the calculation time.

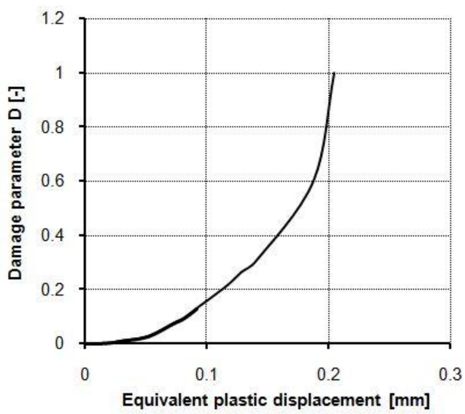
Three dimensional 4-node linear tetrahedron elements (C3D4) are chosen for headed stud and concrete parts. For the steel profile, the 8-node hexahedron solid element (C3D8) is chosen. Size of the finite elements was set to approximately 2 mm in zones around shear connectors and increased to 10 mm towards the outer edges of the model. Monotonic load is applied as displacement control at the top cross section of steel profile. The general contact interaction with 0.24 friction coefficient was defined for the Explicit solver to handle the complex contact conditions between all the parts in the FE model. No cohesive behavior was accounted in the model since the contact between the steel flange and the concrete slab was greased during the experiments.

Material properties obtained in standard material tests were used for calibration of the material models of steel and concrete in ABAQUS. The isotropic plasticity model was used for the headed studs, steel profile and the reinforcement bars. Damage material models, namely the ductile damage and the shear damage models offered in ABAQUS (2013), were used to model fracture of the headed studs.

Damage variable *D* is used to define degradation of stiffness after the damage initiation, see Fig. 5(b). Up to the point of damage initiation, the value of damage variable is equal to $D = 0$. Damage variable value of $D = 1.0$ corresponds to the failure of headed studs' material with equivalent plastic displacement calibrated to value of 0.204 mm for the given finite element size and type. According to Pavlovic *et al*. (2013), damage variable is in relation with



(a) Damage initiation criterion



(b) Damage evolution law

Fig. 5 Parameters of damage material model for headed stud material

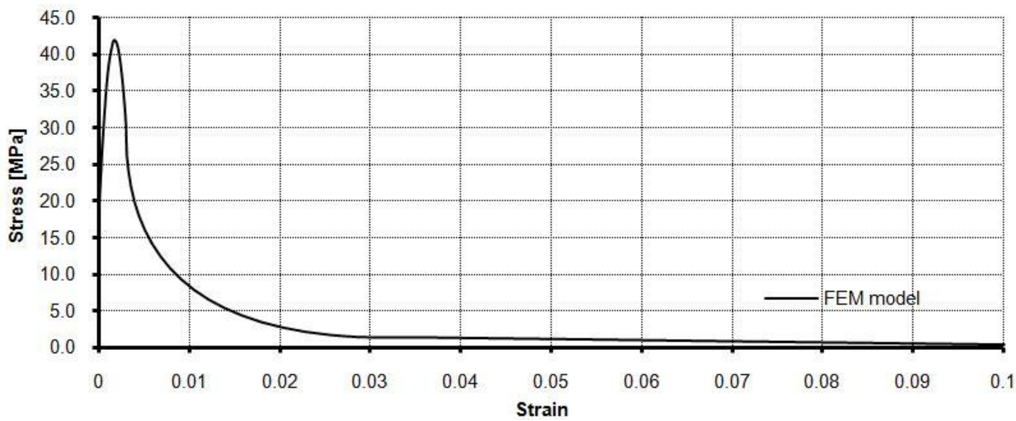


Fig. 6 Stress–strain curve for concrete in compression

the element size. Element size of 1.2 mm is adopted for the headed stud. For concrete part of the model, element size of 2.4 mm in contact zone with the stud is adopted. For shear damage variable, the displacement at failure of 0.4 mm was used. Simulation of tensile coupon test was used for calibration of parameters of the headed stud's material model. Comparison of stress – strain curve tests vs. FE model for headed stud steel and the steel profile is shown in Fig. 4.

Concrete damage plasticity model was used for the concrete part of the specimen. The stress-strain curve shown in Fig. 6, was defined according to Eurocode 2 (2004) for the part up to the strain of ε_{cu1} = 3.5 ‰ with f_{cm} = 42 MPa and modulus of elasticity E_{cm} = 33000 N/mm^2. For the strain ε_c > 3.5‰, the stress-strain curve for concrete is defined according to Pavlovic *et al*. (2013) as a combination of sine function and linear function, using the following coefficients α = 20, α_{tE} = 1.05, α_{tD} = 0.45. Concrete damage variable for compression has value D = 0 in the stress domain up to the inelastic concrete strain of 0.0014. For the inelastic strain of 0.1, the damage variable has value 0.995 in order to avoid numerical instability of the solution, see Fig. 7. Parameters of concrete behavior in tension were defined by separate stress-strain and damage curves. The ultimate tensile strength f_{ctm} = 3.0 MPa.

The FE model was validated by comparison to the load-slip curves from experiments for several layouts of group arrangements and stud diameters, Spremic *et al*. (2013). The final calibration comprised of choice of unique set of material models and boundary conditions parameters for all analyzed cases that gives minimum error of the FE model. Examples of the excellent agreement between FEA results and load-slip curves from experiments are given in Figs. 8(a) and (b). Elastic behavior, plastic behavior and fracture and are well predicted by the FE model.

4. Failure modes of studs in group arrangement

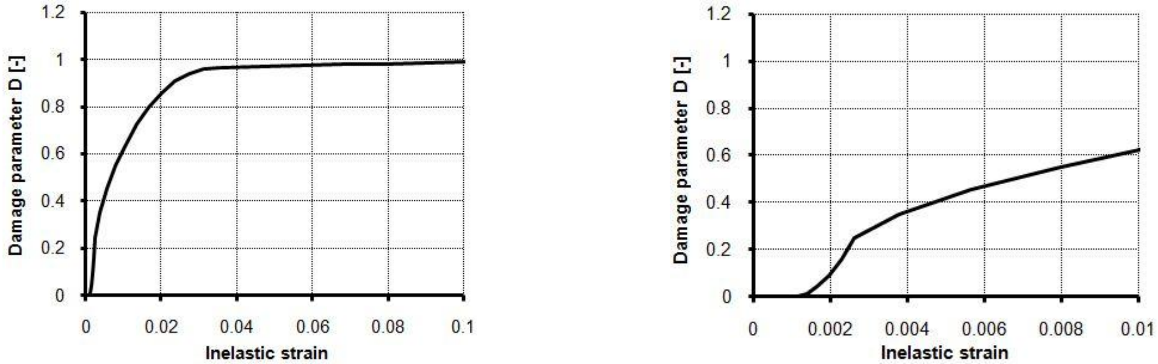
There are three possible failure modes in the shear connection by headed studs:

- fracture of the headed studs,
- concrete cone failure and
- the combined failure mode.

The range of governing of these failure modes in the grouped arrangements of headed studs in the slab pocket are described below.

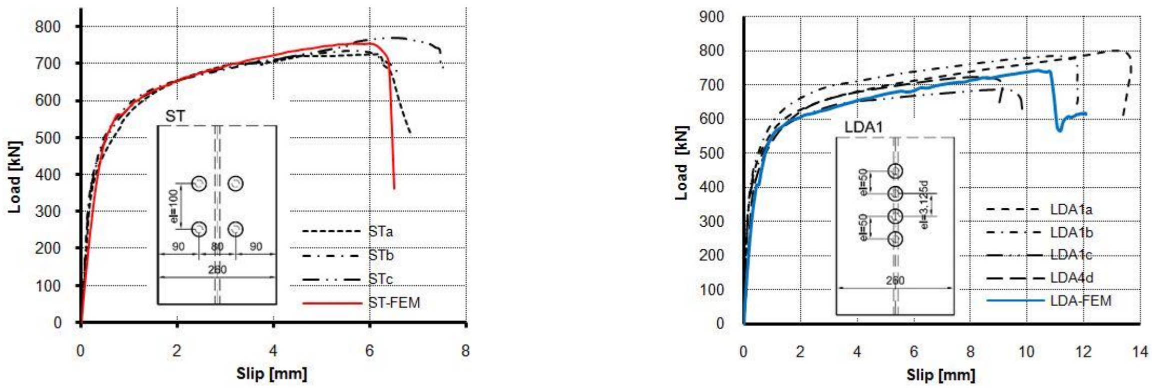
4.1 Headed stud failure mode

Strut and tie components are used to explain several load transferring mechanisms of shear forces in reinforced concrete structures. The same components are recognized



(a) Entire strain range (b) Strains up to 1%

Fig. 7 Concrete Damage Plasticity model - damage evolution law in compression



(a) Specimens ST (b) Specimens LDA

Fig. 8 Load-slip curves: Experiments vs. FEA by analyzing distribution and directions of principal stresses obtained from FE results. Ties are presented by the headed studs, while struts are formed by diagonally oriented compressive stresses in the concrete surrounding the studs, see Fig. 9. In order to establish the strut-tie model, it is necessary that the headed studs are adequately anchored in the concrete having sufficient tensile resistance of the studs and pull-out resistance of the concrete.

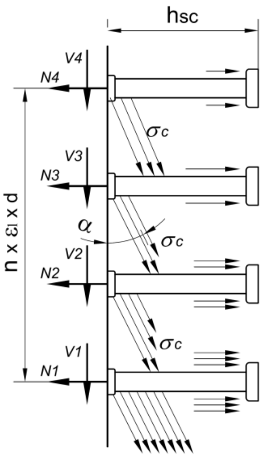


Fig. 9 Strut and tie model

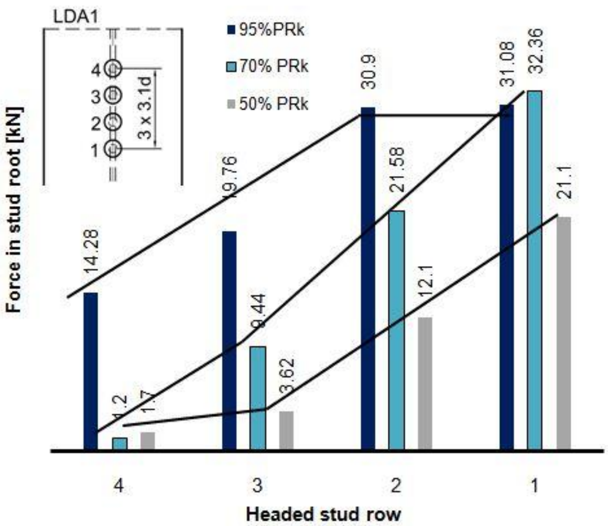


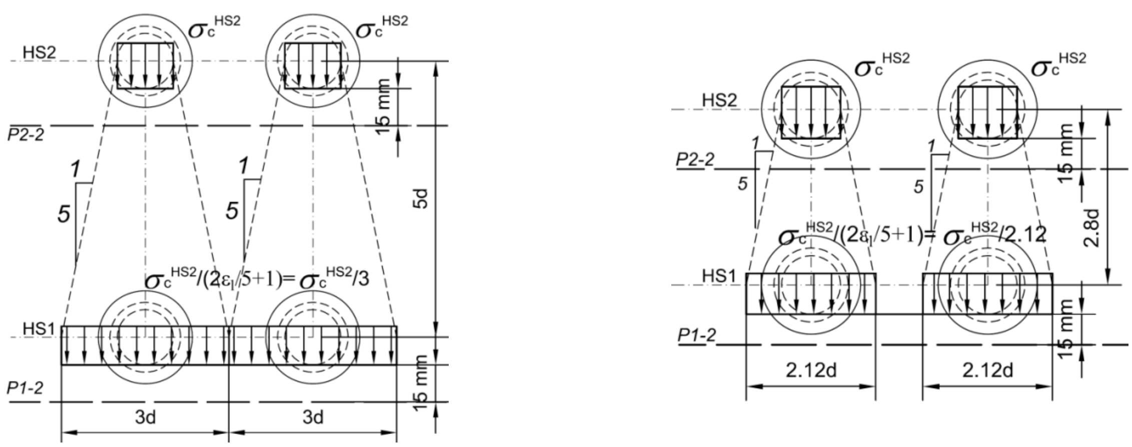
Fig. 10 Tensile force in the root cross section

Fig. 10 shows development of the tensile forces at the headed stud roots during the push-out tests obtained from FEA results. Resistance of the cross section of the stud is governed by the interaction of internal forces: shear force, tensile force and the bending moment. For values higher than 70% of the ultimate shear force, the tensile force in the first headed stud remains constant, Fig. 10. If the headed studs have a necessary length to realize adequate anchoring into an RC slab, the failure occurs due to a headed stud failure, irrespective of the distance between adjacent headed studs in the group. Constant value of the tensile force is the result of onset of yielding of the headed stud cross section. Shear resistance depends on the relation between shear force and tensile force.

The intensity of the tensile force in the headed studs depends on the inclination of the compressed diagonal, strut, in the concrete. The inclination of compressed diagonal in concrete is a function of two parameters: distance between headed studs and compressive strength of the concrete. Distance between studs has low influence to shear resistance of stud cross section.

4.2 Concrete cone failure mode

In order to define the impact of the failure of concrete in front of the headed studs on the shear resistance of the group, the values of stress in concrete in front of the headed studs are analyzed, see Figs. 11(a) and (b). The stresses in concrete of two groups of headed studs in 2×2 layout were compared. In the first group, standard arrangement was used with the distances between the headed studs equal to 5*d*, see Fig. 11(a). In the second group GR1, shown in Fig. 11(b), the longitudinal distance between the headed studs was reduced to 2.8*d*. The transverse distance between the studs within a row is also reduced in the case of group GR1. The stress values in front of the headed studs will be analyzed. Compressive stress propagates from the contact surface between the headed studs and the concrete slab to all directions. The angle of distribution of local stress in



(a) Group of headed studs with the standard distance

(b) Group of headed studs with reduced distance

Fig. 11 Distribution of bearing stresses

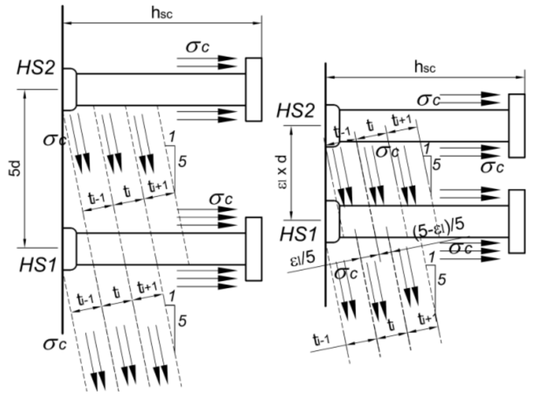


Fig. 12 Strut and tie model – Bearing stress in concrete

concrete, 1:5 inclination, is assumed according to Eurocode 2 (2004). If there is more than one compression force acting on the concrete cross section, the distribution areas are overlapping at certain distance. For the groups shown in Figs. 11(a) and (b) with distances 5*d* and 2.8*d*, respectively, it can clearly be seen that there is no overlapping of the distribution areas.

Fig. 12 illustrates bearing stress propagation in the vertical cross section through the group. Thickness of the concrete layer is t_i. It is clear from Fig. 12 that the layer t_1 in front of the headed studs HS1 and HS2 does not overlap in the case of standard group of studs. Index *i* determines the layer number. The first layer $i = 1$ is the layer in contact with steel profile. The value of stress in front of the first headed stud HS1 in a group, based on the stress distribution presented in Figs. 11(a) and 12, equals

$$\sigma_{t_i} = \sigma_{t_i}^{HS1} + \sigma_{t_{i-1}}^{HS2} / 3 \tag{1}$$

In the previous expression, the stress from the layer $i - 1$ is divided by the width of the distribution area, see Fig. 11(a). Distribution area has the width of $2\varepsilon_l / 5 + 1 = 3$ for studs' distance equal to 5*d*, see Fig. 11(a). In the case of the group, when the headed studs are at a distance lower than 5*d*, there is an overlapping of the t_{ci} layers in front of the headed studs in the first HS1 and the t_i layer in front of headed studs in the second row HS2. According to bearing stress presented in Fig. 12, the following equation for stress in layer t_i in front of first stud in group is valid

$$\begin{aligned}\sigma_{t_i} &= \sigma_{t_i}^{HS1} + \frac{\varepsilon_l}{5}\sigma_{t_{i-1}}^{HS2} / \left(2\varepsilon_l / 5 + 1\right) \\ &+ \frac{5-\varepsilon_l}{5}\sigma_{t_i}^{HS2} / \left(2\varepsilon_l / 5 + 1\right)\end{aligned} \tag{2}$$

In the previous expressions:

- HS1, HS2 are the designations of the headed studs in a group according to figures
- t_i is the thickness of a "layer“ of concrete according to Fig. 12,
- σ_{t_i} is the compression stress in a layer “i” in concrete in front of the headed stud group
- $\sigma_{t_i}^{HSj}$ is the contact compression stress in a layer “i” in concrete in front of the headed stud “j”
- ε_l is normalized longitudinal distance of the headed studs, i.e., the distance divided by the diameter of the studs (e_l/d).

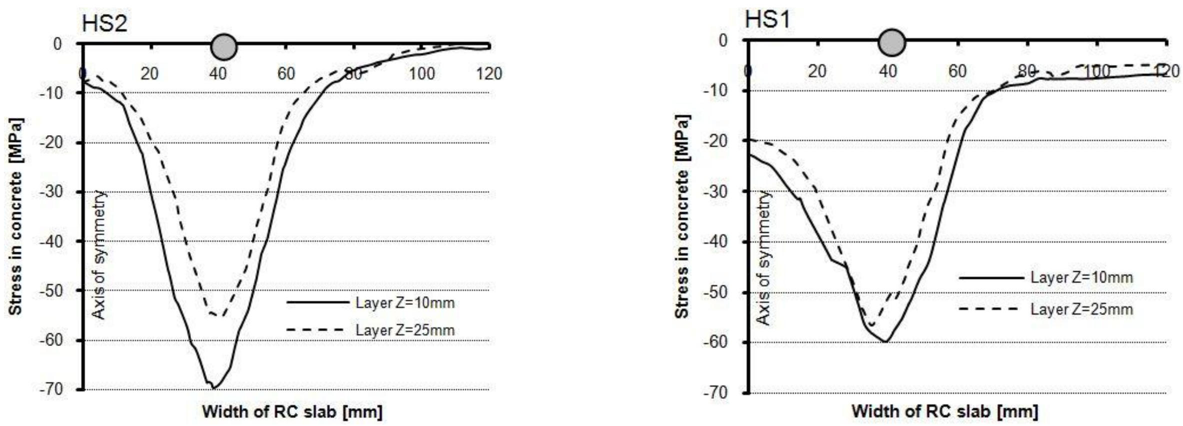
For longitudinal distance, equal to 5*d*, Eq. (2) has the same value as Eq. (1). By comparing the expressions (1) and (2), it can be concluded that the stress in the concrete layers in front of the first stud in group HS1 is higher in the case when the distances between the headed studs are reduced (< 5*d*). The stresses in front of the headed studs HS1 and HS2 were determined from the FEA results. The stresses are determined for two layers of concrete, at 10 mm and at 25 mm above the steel flange. The results are presented in Figs. 13 and 14.

Stress diagrams in front of HS1 and HS2 for the standard group arrangement are very similar. Stresses in the concrete in front of headed studs HS1 and HS2 are almost equal for layer z = 10 mm and z = 25 mm. The stress differences are in the range of up to 15% for layer z = 10 mm. In layers z = 25 mm, the stresses are equal for the first and the second stud. Superposition of stresses can be seen in the zone between the studs in the first row, where the value of stresses of 20 MPa is twice larger compared with stress between studs HS2. This confirms that in the case of standard arrangement of studs there is no superposition of the same layer of stresses in front of HS1 and HS2.

Diagrams of stresses in front of studs HS1 and HS2 in group arrangement with reduced distance (GR1) are different in values and forms compared to the standard arrangement (ST). Superposition of bearing stresses in concrete is quite evident in the zone between studs in front of HS1. The most significant differences are between stresses in layer z = 10 mm and z = 25 mm. Stresses in layer z = 25 mm are 50% larger compared with values of stresses in layer z = 10 mm. Up to 100% higher concrete stresses are in front of the group compere with stresses in front of group ST, especially in the higher layers of concrete.

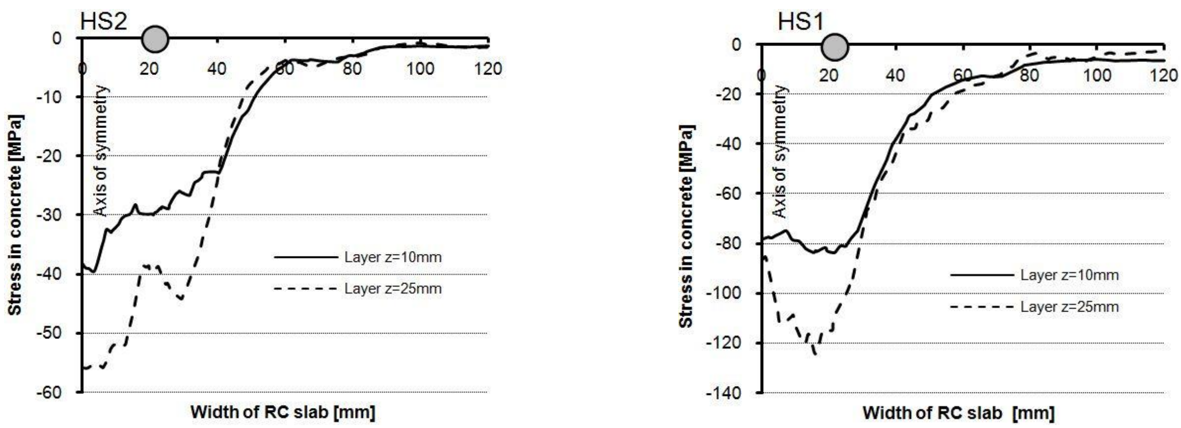
Reduction of the distance between the headed studs in the direction of the shear force results in the change of direction of compressed diagonal in concrete, and the value of compression stress in the concrete slab in front of the first headed studs. Based on the FE results, it was concluded that the higher layers of concrete are engaged for transfer of the shear force, in cases where the headed studs are at the reduced distances. This shift of the resultant force acting on the headed studs, see Fig. 15, causes larger bending of the studs and therefore larger deformations of the concrete in the stud root. Substantial crushing of the concrete around the root of the studs and bending of the studs causes the change of the direction of contact forces at the surfaces between the headed studs and concrete. The angle of contact forces resultant depends on the on the stresses in the concrete slab and the distance between studs in force direction. Based on this finding, one could find the relation between the stud distance and the angle of the contact forces resultant.

Change of the resultant force direction causes tensile force in the direction of the headed stud which tends to pull the headed stud out of concrete, Spremic *et al*. (2017a). To prevent concrete failure mode, it is necessary to use higher studs to prevent crack initiation in the concrete. As long as



(a) ST - Section P2-2 (see Fig. 11(a)) (b) ST - Section P1-2 (see Fig. 11(a))

Fig. 13 Bearing stresses in concrete (FEA results) – standard group arrangement (ST)



(a) GR1 - Section P2-2 (see Fig. 11(b)) (b) GR1 - Section P1-2 (see Fig. 11(b))

Fig. 14 Bearing stresses in concrete (FEA results) – group arrangement with reduced distance (GR1)

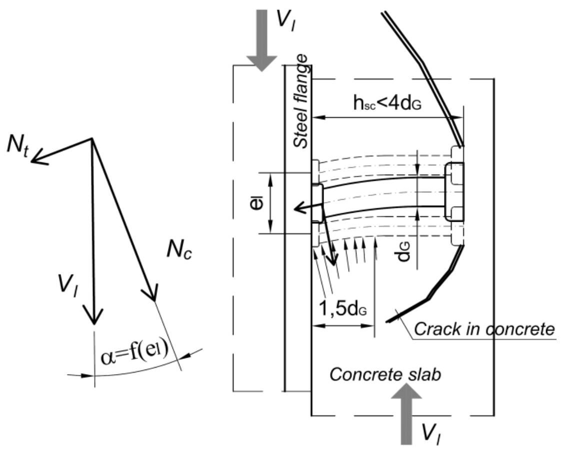


Fig. 15 New calculation model based on the equivalent stud

there is a confined condition in the concrete around the headed studs, the bearing resistance of concrete is apparently not critical for the level of stresses in concrete imposed by the load transfer. It is well known that due to the confined stress condition in the zone of the concrete slab immediately around the studs, the values of stress considerably higher (up to 10 times) than the characteristic compressive strength of concrete can occur, as shown by Pavlović *et al*. (2013) and Oehlers (1980).

To sustain the confined condition of stress in concrete, the minimum necessary headed studs' height can be defined. This minimum required studs' height also needs to prevent initiation of characteristic cracks in concrete caused by tension force, see Fig. 15. Tensile force occurring in the connection needs to be adequately anchored in an RC slab.

5. The equivalent stud diameter model

When the height of studs in group arrangement is not adequate to prevent concrete failure mode, initiation of a crack in the slab, the behavior of a stud group is very similar to the behavior of short headed studs, $h_{sc} < 4d$ according to EN 1994-1-1 (2004). This fact is used for derivation of a new calculation model for shear resistance of closely spaced group of headed studs. A group of headed studs is approximated using an equivalent headed stud of the same height as individual headed stud, but having a larger, equivalent, diameter. A new model for shear resistance calculation of a group of headed studs is based on the ratio of the headed stud height and the newly introduced parameter – the equivalent diameter of the studs group d_G, see Fig. 15.

A detailed description of the calculation models is presented in Spremic *et al*. (2017b). An equivalent stud has the same height as a single stud, but it is larger in diameter in comparison with the single headed stud. The diameter of the equivalent headed stud can be calculated by multiplying the single stud's diameter d with the parameter m and the number of columns of studs in the group n_c as follows

$$d_G = d \cdot (1+m) \cdot (0.9 + n_c / 10) \tag{3}$$

The parameter m is the function of a number of rows n_r of headed studs in the group and distance between the rows $\varepsilon_l = e_l/d$, and it is calculated as

$$m = n_r - n_r^{\varepsilon_l/5} \tag{4}$$

Previous Eq. (3) for the headed studs with a distance equal or larger than 5d results in an equivalent diameter equal to a single stud diameter in cases with one column of studs. In cases of headed stud' groups, when the distance between the studs in force direction is $3d \le e_l \le 5d$, it is necessary to reduce the shear resistance. The reduction was defined with the reduction factor α_G for the shear resistance of a stud group. In this case, the shear resistance of the group of headed studs can be determined as follows

$$P_{Rk,G} = \alpha_G \cdot n_r \cdot n_c \cdot P_{Rk} \tag{5}$$

In Eq. (5), n_r is the number of rows (in the longitudinal direction) and n_c is the number of columns in the group, see Fig. 1. Behavior of the stud group with reduced distance between studs in force direction is similar to the behavior of single short headed stud which reduction of shear resistance is governed by the reduction coefficient given in EN 1994-1-1 (2004). Taking into account the newly introduced parameter, the equivalent diameter of the group of studs, the reduction factor for group behavior can be calculated using the equation given in Eurocode 4 (2004) as follows:

$$\alpha_G = \min\left(k\left(\frac{h_{sc}}{d_G} + 1 \right), 1 \right) \tag{6}$$

Previous Eq. (6) is the same as the one from EN 1994-1-1 (2004) with two changes:

- d_G is the newly introduced parameter – equivalent diameter of the group of studs instead of diameter for a single stud,
- k is a new parameter instead of constant value 0.2 according to EN 1994-1-1 (2004).
- h_{sc} is the overall height of stud after the welding.

The value of parameter k is in function of the diameter of the single headed studs

$$k = \min\left\{ 0.2\,;\, 0.2 \cdot \frac{20}{d} \right\} \tag{7}$$

Recommendation for shear resistance design is applicable for groups of studs with distance between studs, in direction transverse to the shear force, equal or larger than 2.5d. The values of the reduction factor for various layouts of groups are presented in Spremic *et al*. (2017b) together with the comparison of the proposed model to the results of existing experiments by other researchers.

6. Parametric study

The focus of this study is on the longitudinal shear connection in typical steel-concrete composite floor beams in commercial and office buildings. Geometry and shear resistance of stud's group, number and diameter of studs in group, for parametric study was chosen to be appropriate for typical composite floor beam with span up to 15.0 m. FE model validated upon push-out experiments is used for the parametric study analyzing influence of the diameter d of the headed stud and the layout of the stud group arrangement on the shear resistance and slip capacity of the shear connection. The range of parameters is presented in Table 2.

6.1 Influence of the headed stud diameter

Shear resistance of a group of four headed studs having

Table 2 Parameters used in FE models

Group mark	f_{cm} [MPa]	d [mm]	h_{sc} [mm]	n_cx n_r	ε_t	ε_l
GR1-16	42.0	16	100	2×2	2.8	2.8
GR1-19	42.0	19	100	2×2	3.15	3.15
GR1-12	42.0	12.7	100	2×2	3.07	3.07
GR 33	42.0	16	100	3×3	3.1	3.1
GR 33 h_{sc}=140 mm	42.0	16	140	3×3	3.1	3.1
GR 23	42.0	16	100	2×3	3.1	3.1
GR 23 h_{sc}=140 mm	42.0	16	140	2×3	3.1	3.1
GR 32	42.0	16	100	3×2	3.1	3.1
GR 32 h_{sc}=140 mm	42.0	16	140	3×2	3.1	3.1

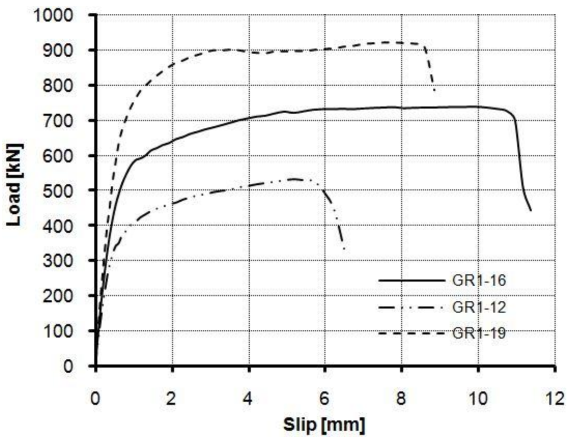


Fig. 16 Load-slip curves, group of four studs, FEA results

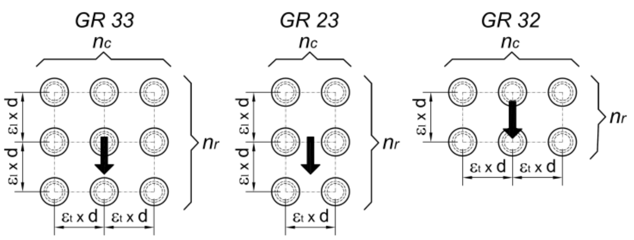


Fig. 17 Layout of analyzed groups

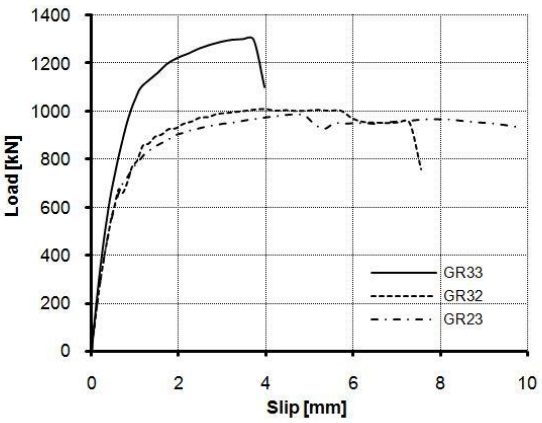


Fig. 18 Load – slip curves, FEA results

diameter 12.7 mm, 16 mm and 19 mm was analyzed. By varying the headed stud diameter, and keeping the same height of headed studs, the influence of height-to-diameter ratio h_{sc}/d on the shear resistance of the group is considered. Load–slip curves obtained from FEA results are presented in Fig. 16. Comparison of the results of FEA parametric study and values of reduction coefficient according to the proposed calculation model based on the equivalent diameter are presented in Table 3.

Eurocode 4 limits the application of the expression for headed studs' shear resistance to the headed studs having diameter 16-25 mm. The obtained values of numerical analysis, of the group of headed studs having diameter of 12.7 mm, were compared to experimental results provided by Xu *et al*. (2012). The presented experimental results in Xu *et al*. (2012) and the results of numerical analysis are identical. The differences of the proposed reduction coefficient and the results of the numerical model are up to 5%, whereby the proposed reduction coefficient in all the presented cases is on the safe side, see Table 3. Characteristic shear resistance of headed studs $P_{Rk,EC}$, in Table 3 was calculated, in accordance with Eurocode 4, with experimental values of materials properties (Figs. 4 and 6).

6.2 Influence of the group arrangement

The FEA included three layouts of groups with headed studs having diameters of 16 mm and height of 100 mm. Group geometry was presented in Table 2 and in Fig. 17. The analysis of these groups included influence of dimensions, layout and number of headed studs in a group on the group's shear resistance. The GR33 group with nine headed studs is the group which is the most frequently analyzed group in literature. The GR23 and GR32 are groups with six headed studs each which could be used in prefabricated composite beams in building construction, in cases when high capacity shear connection is required, for instance, for the composite beam spans exceeding 15 m. The obtained results, load – slip diagrams are presented in Fig. 18.

Reduction coefficient values which are the result of the numerical model and calculation according to the proposed model are presented in Table 3. The value of the reduction coefficient, which is based on the equivalent headed stud diameter, is by 1-8% different from the results of the numerical model. The difference in results in all the cases is on the safe side. Slip at failure of GR33 group is less than 6 mm. The characteristic failure at ultimate load of this group is concrete failure. These facts confirm the analogy between the behavior of a stud' group and behavior of a short single stud. It can be expected to obtain such results in cases of groups of headed studs with ratio $h_{sc}/d_G < 3$. Considering the mentioned analogy, a recommendation of a minimum h_{sc}/d_G ratio could be made, so that the connection would be considered ductile.

To verify the mentioned assumption about group ductility, in case of the groups GR33, GR23 and GR32, an additional numerical model was made. The goal is to verify the group ductility in function of the ratio of headed stud height and the equivalent headed stud diameter. The groups GR33, GR23 and GR32 were analyzed with the headed

Table 3 Reduction of the shear resistance of headed studs in the group arrangement: FEA results vs. the equivalent diameter model

Layout	d [mm]	m	d_G [mm]	h_{sc}/d_G	α_G	$P_{FEA,stud}$ [kN]	$P_{Rk,EC4,stud}$ [kN]	$\alpha_{G,\ FEA}$	$\alpha_G / \alpha_{G,\ FEA}$
GR1-16	16	0.515	26.67	3.74	0.95	92.4	95.6	1.00	0.95
GR1-19	19	0.452	30.35	3.30	0.85	115.5	134.8	0.86	0.988
GR1-12	12.7	0.46	19.39	5.15	1.00	66.8	53.8	1.00	1.00
GR33	16	1.02	38.9	2.50	0.72	72.2	95.6	0.76	0.947
GR32	16	0.46	28.1	3.60	0.91	87.5	95.6	0.92	0.99
GR23	16	1.02	35.6	2.81	0.77	80.2	95.6	0.84	0.92
GR33-hsc = 140 mm	16	1.02	38.9	3.59	0.91	82.2	95.6	0.87	1.04
GR32-hsc = 140 mm	16	0.46	28.1	4.98	1.00	96.2	95.6	1.00	1.00
GR23-hsc = 140 mm	16	1.02	35.6	3.93	0.98	93.0	95.6	0.98	1.00

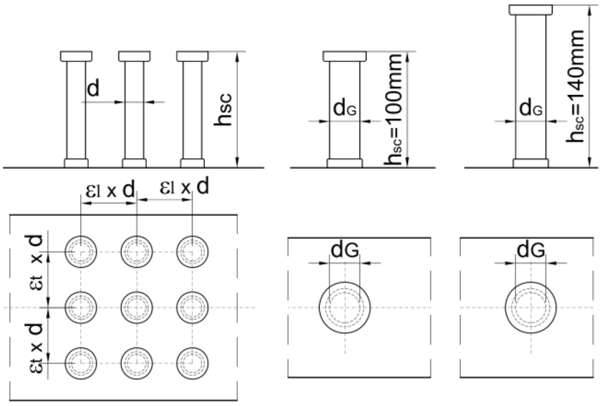


Fig. 19 Equivalent diameter of the group of headed studs - case GR33

studs having diameter 16 mm and headed stud height h_{sc} = 100 mm and h_{sc} = 140 mm, see Table 2 and Fig. 19. Due to the increased headed stud height, concrete slab was modified, and a thicker one, 160 mm was adopted. The reinforcement in the slab, as well as other geometrical characteristics, is the same as in all presented FE models.

In the case of the group of nine headed studs having diameter 16 mm, constructed at a mutual distance of 3.1*d* (50 mm), in the direction of force and perpendicular to the shear force direction, according to the proposed model, the equivalent diameter of the group is equal to

$$m = n_r - n_r^{\varepsilon_l/5} = 3 - 3^{3.1/5} = 1.02 \tag{8}$$

$$\begin{aligned} d_G &= d(1-m)\cdot(0.9+n_c/10) \\ &= 16\cdot(1+1.02)\cdot(0.9+3/10) = 38.9\,\text{mm} \end{aligned} \tag{9}$$

For the adopted height of the headed studs of 140 mm, the ratio of the height and equivalent diameter of the headed studs is h_{sc}/d_G = 3.53. Based on the previously presented results, it can be expected that such height of a group of headed studs would result in ductile behavior of the connection. The obtained results, load – slip curves of the numerical models are presented in Figs. 20(a) and (b).

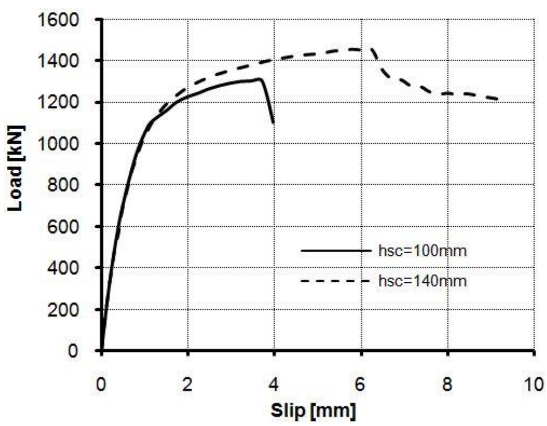
The slip of the longitudinal shear connection of the groups GR33, GR23 and GR32 with 140 mm high headed studs at ultimate load is 6.3 mm, 11.6 mm and 7.6 mm respectively. On the basis of this result, the connection can be characterized as ductile. The obtained values of the reduction coefficient, through a numerical model and proposed design in this paper were presented in Table 3 and in Figs. 20(a) and (b).

The obtained results confirm the fact that the groups with higher headed studs also have higher values of ultimate shear resistance. Also, adoption of higher headed studs can provide ductile behavior of a connection created with a group of headed studs. The obtained values of reduction coefficient according to the FEA and according to the proposed design model are in good agreement. Good prediction of shear connection ductility can be provided based on the ratio h_{sc}/d_G.

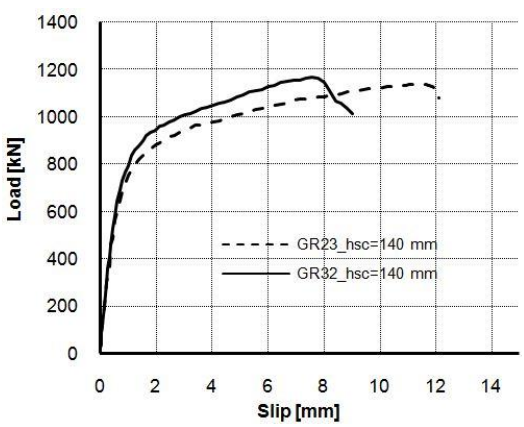
7. Conclusions

Shear resistance and slip capacity of headed stud shear connectors in various closely spaced group arrangements (e_l/d < 3.7) are obtained in push-out experiments. Finite Element (FE) models incorporating complex contact interactions and material damage are used to validate the experiment results. Based on the FEA validation and parametric study, where the size effects of the group are analyzed by varying the layout of the group: 4 to 9 studs, the diameter of the studs: 12 mm to 19 mm and height of the studs: 100 mm and 140 mm, the following conclusions are drawn at the ultimate load level:

- Bearing stresses in concrete are increased approximately 100% in case of closely spaced group compared to standard arrangement of headed studs, 2.8*d* and 5*d*, respectively, due to the overlapping of the stress distribution areas. The increased bearing stresses influence the shift of the force resultant towards the stud head and therefore the larger deformations in the concrete.
- Pry-out failure in the concrete for the group of headed studs is prevented by a sufficient height of



(a) FEA results GR33



(b) FEA results GR23 and GR32

Fig. 20 Load – slip curves, FEA results

the studs. The height of 8*d* and 10*d* is needed for closely spaced groups of 4 and 9 studs, where the distance between studs is equal to 3*d*.

- The calculation model for the shear resistance reduction of closely spaced headed studs is based on the equivalent diameter model. The model gives excellent prediction of the results obtained in push-out experiments and FE parametric study. Predictions of the shear resistance given by the equivalent diameter model are always conservative with maximum scattering of 8%.
- There is no reduction of the shear resistance due to the group behavior if the ratio between the stud height and the equivalent diameter is higher than $h_{sc}/d_G > 4$. The behavior is ductile according to EN 1994-1-1 (2004) in the range of parameters analyzed in this study.

Acknowledgments

The research described in this paper was supported by the Serbian Ministry of Education, Science and Technological Development through the TR-36048 project.

References

ABAQUS (2013), Documentation 6.11; Dassault system.

An, L. and Cederwall, K. (1996), "Push-out tests on studs in high strength and normal strength concrete", *J. Constr. Steel Res.*, **36**(1), 15-29.

ANSI/AISC 360-05 (2005), An American National Standard, Specification for Structural Steel Buildings; American Institute of Steel Construction, Chicago, IL, USA.

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

Eurocode 4, EN1994-1-1 (2004), Design of composite steel and concrete structures. Part 1-1: General rules and rules for buildings, European Committee for Standardization; Brussels, Belgium.

Eurocode 4, EN1994-2 (2008), Design of composite steel and concrete structures. Part 2: General rules and rules for bridges, European Committee for Standardization; Brussels, Belgium.

Guezouli, S., Lachal, A. and Nguyen, Q.H. (2013), "Numerical investigation of internal force transfer mechanism in push-out tests", *Eng. Struct.*, **52**, 140-152

Oehlers, D. (1980), "Stud shear connectors for composite beams", Ph.D. Thesis; Department of Engineering, University of Warwick, March 1980.

Okada, J. and Lebet, J.P. (2000), "Push-out tests for grouped arrangement of stud connectors", Report No ICOM 420; EPFL-ICOM Lausanne, September 2000.

Okada, J., Yoda, T. and Lebet, J.P. (2006), "A study of the grouped arrangements of stud connectors on shear strength behavior", *Struct. Eng./Earthq. Eng.*, **23**(1), 75-89.

Pallarés, L. and Hajjar, J.F. (2010), "Headed steel stud anchors in composite structures, Part I: Shear", *J. Constr. Steel Res.*, **66**, 198-212.

Pavlović, M., Marković, Z., Veljković, M. and Buđevac, D. (2013), "Bolted shear connectors vs. headed studs behaviour in push-out tests", *J. Constr. Steel Res.*, **88**, 134-149

Shim, C.S., Lee, P.G., Kim, D.W. and Chung, C.H. (2008), "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 Conference*, Tabernash, CO, USA, July.
DOI: 10.1061/41142(396)8

Spremic, M., Markovic, Z., Veljkovic, M. and Budjevac, D. (2013), "Push-out experiments of headed shear stud in group arrangement", *Adv. Steel Constr., Int. J.*, **9**(2), 170-191.

Spremic, M., Markovic, Z. and Veljkovic, M. (2017a), "Recommendations for the design of grouped headed studs", Steel Construction – Design and Research, **10**(2), 145-153

Spremic, M., Markovic, Z., Veljkovic, M. and Budjevac, D. (2017b), "Shear resistance design of headed studs in group arrangement", Engineering Structures 2017. [In Review]

Xu, C., Sugiura, K., Wu, C. and Su, Q. (2012), "Parametrical static analysis on group studs with typical push-out tests", *J. Constr. Steel Res.*, **72**, 84-96.

Xue, D., Liu, Y., Yu, Z. and He, J. (2012), "Static behavior of multi-stud shear connectors for steel-concrete composite bridge", *J. Constr. Steel Res.*, **74**, 1-7.

DL