ГОДИШНИК НА УНИВЕРСИТЕТА ПО АРХИТЕКТУРА, СТРОИТЕЛСТВО И ГЕОДЕЗИЯ СОФИЯ

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EXPERIMENTAL INVESTIGATION OF LOAD AND SLIP CAPACITY OF HEADED STUD CONNECTORS IN COMPOSITE SLABS FOR BUILDINGS

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Keywords: *composite structure, shear connector, push test, EC-4*

ABSTRACT

A comprehensive experimental investigation in accordance with EC4-1-1 Annex B.2 is performed by the Department of Metal Structures at the Civil Engineering Faculty – Skopje. Twenty-nine specimens are investigated, with variation of constructive parameters, with push test. Types of steel sheeting and position, longitudinal and transversal, are variated for headed stud connector $d = 19$ mm, $h_{sc} = 100$ mm. Also, a number of shear connectors in cross-section (1 or 2), and welding procedure, with holes and through deck welding, are variated. Concrete with grade C25/30, as a commonly used grade, and reinforcement with minimum percentage, are adopted. Results of load capacity are compared with proposed load capacity according to EC4-1-1 (6.6.3 and 6.6.4), where significant differences in some cases are obtained.

This paper presents the preparation and performance of the experimental investigation. Also, the obtained load and slip capacities are presented. The shape of fracture and ductility are subjects of investigation in order to establish analytical model for the shear connection.

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1. Introduction

Composite steel and concrete structures are extensively used in building high-rise buildings, administrative buildings and bridges all around the world. These structures offer fast, economical and eco-friendly construction. The possibility of mastering large spans with beams and deck with relatively small dimensions, lighter structure for 20 – 40% and faster construction, made these structures very popular among the architects and civil engineers.

Unfortunately, aside of all worldwide positive experiences, composite steel and concrete structures are occasionally used in construction works in our country. Possible reasons are lack of experience and traditional concrete oriented construction.

Floor slabs at buildings are a field where advances of composite steel and concrete structures are evident. The common concrete slab positioned on top of steel beam (usually welded or rolled I or H steel section) opens the door for composite action. The concrete slab could be cast on traditional formwork or profiled steel sheeting could be used as formwork. A second case, mostly used, offers fast building and possibility to establish composite action in flor slab (composite slab). However, the main benefit is reached with composite action of steel beam and concrete slab.

The transfer of longitudinal shear between the steel beam and the concrete slab is achieved by installing various types of mechanical devices called shear connectors. The most used shear connectors at composite slabs for buildings are headed studs (Fig. 1). Headed studs, in case of profiled steel sheeting, could be welded through a hole in steel sheeting, or welded through the deck (Fig. 2).

Figure 1. Headed stud shear connectors

Figure 2. Welding of headed stud

Shear connectors shall have sufficient resistance to transmit the shear force, and deformation capacity to justify any inelastic redistribution of assumed shear in design. Therefore, it is necessary to determine the shear resistance and slip capacity of the headed stud connectors prior to their use in construction.

Eurocode 4 [1] prescribes the principles and rules for design of composite steel and concrete structures. Design resistance of headed stud connectors in solid slabs and concrete encasement is stated in clause 6.6.3 and clause 6.6.4 when headed studs are used with profiled steel sheeting. Also, annex B.2 gives the rules for a test of shear connectors.

There are many types of profiled steel sheeting with open and re-entrant profile used in composite slabs. Design resistance and ductility of headed stud connectors depend on many parameters: type of steel sheeting, position over steel beam (transversal/longitudinal), height and number of connectors in cross-section (1/2), method of welding, depth of slab, quality of concrete, reinforcement, and mutual position of elements in cross-section.

According to our experience, design resistance and rules for ductility proposed by EC4 not always correspond with real resistance and ductility. The values of reduction factors k_l and k_t in some cases are under question. There is a need of experimental investigation to establish more effective compliance between design resistance and real resistance of headed stud connectors.

With such purpose, and in accordance with the process of acceptance and implementation of Eurocodes in our country, experimental investigation of load and slip capacity of headed stud connectors in composite slabs for buildings has been performed by the Faculty for Civil Engineering, Department of Metal Structures at the "Ss. Cyril and Methodius" University – Skopje.

Three types of commonly used (in our country) profiled steel sheeting (HIDECK 75, BONDECK 600 and FR38/158) and headed stud connectors NELSON with diameter $d = 19$ mm and total height $h_{sc} = 100$ mm are adopted. Also, the corresponding solid slabs have been investigated. Concrete grade C25/30 has been adopted, as commonly used concrete quality in floor slabs.

Specific push test in accordance with annex B.2 of EC4-part 1.1 has been carried out on 29 specimens. The preparation of the test specimens, the testing procedure, measurements, and results are presented in this article.

2. Specific Push Test

When the shear connectors are used in T-beams with a concrete slab of uniform thickness, or with haunches, standard push test may be used. In other cases, with longitudinal or transversal sheeting, a specific push test should be used. The specific push test should be carried out so that the slab and the reinforcement are suitably dimensioned in comparison to the beams, according to the rules and recommendations given in EN 1994-1-1, Annex B.2.

2.1. Preparation of Specimens

The length of each slab has been related to the longitudinal spacing of the connectors in the composite steel-concrete structure. The width of each slab was chosen not to exceed the effective width of the slab of the beam. The slab thickness of 120 mm for HIDECK 75 and BONDECK 600 steel sheeting, and 100 mm for FR38/158 steel sheeting was adopted.

The slabs were cast horizontally, as they are cast as part of a composite structure (Fig. 3).

Figure 3. Casting of concrete

The concrete was cast first on one side of the test sample, and with turning, on the other side. The concrete was air-cured as per practice for composite beams.

From each concrete mix four concrete specimens (cubes) were taken for determination of the strength of the concrete for each of the sides of the sample. The concrete specimens were cured alongside the push test specimen.

The yield strength, the tensile strength and the maximum elongation of a representative sample of the shear connector material, steel beam and profiled steel sheeting were determined with the referent standard tests.

2.2. Test Procedure and Evaluation

As recommended in EC4-1-1 annex B.2, the load was applied in increments up to 40% of the expected failure load, and then 25 times cycled between 5% and 40% of the expected failure load. After the 25th cycle, subsequent load increments were imposed up until failure in the specimen is reached, but not in less than 15 minutes. While testing, the longitudinal slip between the concrete slab and the steel beam is measured constantly. Also, the transverse separation between the slab and the steel section was measured as close as possible to each group of connectors.

Expected failure load is obtained by multiplied design shear resistance of a headed stud, according to clause 6.6.3.1 of EC4-1.1 (1), with number of applied connectors.

$$P_{Rd} = \frac{0,8 f_u \pi d^2 / 4}{\gamma_V} \quad \text{or} \quad P_{Rd} = \frac{0,29\alpha d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_V}, \tag{1}$$

whichever is smaller, with $\alpha = 1$ for $h_{sc} / d = 100/19 > 4$.

Partial factor γ_V is taken as 1,0, specified ultimate tensile strength of the material of the stud $f_u = 500$ MPa, the characteristic cylinder compressive strength of the concrete at the age of testing $f_{ck} = 30$ MPa, secant modulus of elasticity of concrete $E_{cm} = 33000$ MPa.

Shear resistance of headed stud with above values in equation (1) is:

$$P_{Rd} = \frac{0,8.500.\pi.19^2 / 4}{1,0} 10^{-3} = 113,35 \text{ kN} \quad \text{or} \tag{2.1}$$

$$P_{Rd} = \frac{0,29.1,0.19^2 \sqrt{30.33000}}{1,0} 10^{-3} = 104,16 \text{ kN}. \tag{2.2}$$

Assumed shear resistance of headed stud is 104,16 kN.

When profiled steel sheeting is used with ribs parallel to the supporting beam, shear resistance should be multiplied by the reduction factor k_l (3), (Fig. 4).

$$k_l = 0,6 \frac{b_0}{h_p} \left(\frac{hsc}{h_p} - 1 \right) \le 1,0. \tag{3}$$

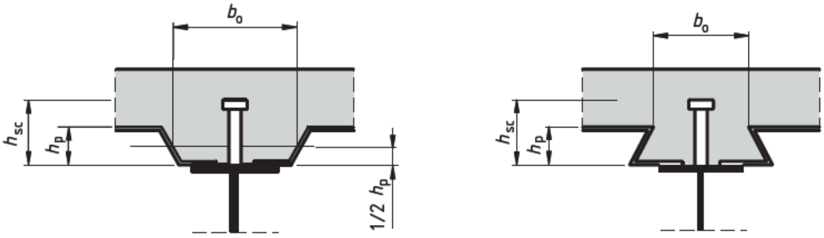

Figure 4. Beam with profiled steel sheeting parallel to the beam

Values of reduction factor k_l , for different types of steel sheeting, are calculated and presented in Part 3.

When profiled steel sheeting is used with ribs transverse to the supporting beam, shear resistance should be multiplied by the reduction factor k_t (4), (Fig. 5).

$$k_t = \frac{0,7}{\sqrt{n_r}} \frac{b_0}{h_p} \left(\frac{hsc}{h_p} - 1 \right). \tag{4}$$

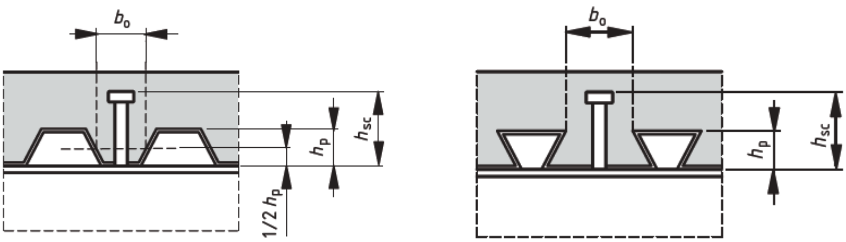

Figure 5. Beam with profiled steel sheeting transverse to the beam

Where n_r is the number of stud connectors in one rib at a beam cross-section.

Upper limit $k_{t,\max}$ for k_t is:

$k_{t,\max} = 0,85$ for $n_r = 1$ and through deck welding;

$k_{t,\max} = 0,75$ for $n_r = 1$ and welding through holes;

$k_{t,\max} = 0,70$ for $n_r = 2$ and through deck welding;

$k_{t,\max} = 0,60$ for $n_r = 2$ and welding through holes.

Values of reduction factor k_t , for different types of steel sheeting, are calculated and presented in Part 3.

2.3. Measuring Equipment

Measuring equipment consists of measuring devices (Fig. 6) and instruments for data acquisition (Fig. 7).

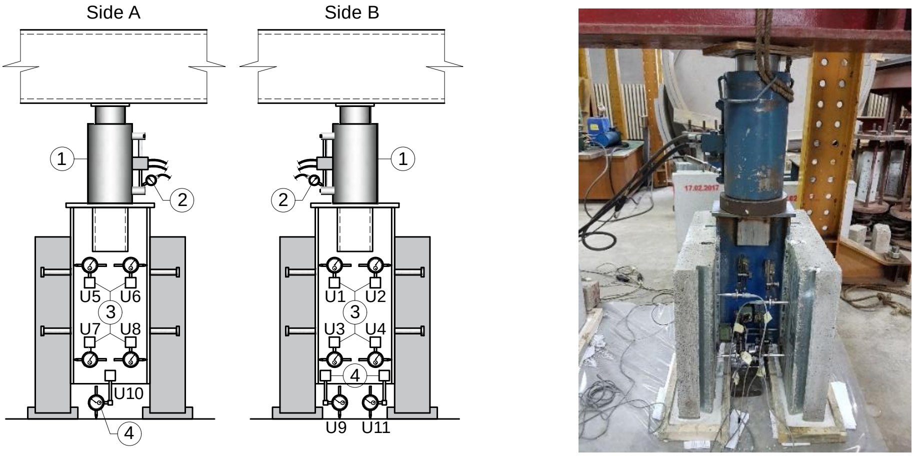

Figure 6. Test equipment – measuring devices

1) 100 tons hydraulic jack; 2) load cell; 3) displacement transducers for measuring the transverse separation between the steel beam and the slabs (U1, U2, U3, U4 on one side and U5, U6, U7, U8 on the other side); 4) displacement transducers for measuring the longitudinal (vertical) slip (U10 on one side and U9, U11 on the other)

Figure 7. Test equipment – data acquisition

The force was applied by 1000 kN hydraulic jack with strain gauge pressure transducer in full bridge. Five strain gauge displacement transducers in full bridge (Kyowa) with measurement range 20 mm were placed on U5, U6, U7, U8 and U10. Four inductive displacement transducers (HBM) with measurement range 10 mm were placed on U1, U2, U3 and U4. Two inductive displacement transducers (HBM) with measurement range 50 mm were placed on U9 and U11.

Measuring devices were connected to data processing instruments HBM Quantum and HBM Spider 8. Two personal computers and program Catman Easy (HBM) for data storage were used. Data acquisition was with frequency of 5 Hz.

3. Description of Specimens

Experimental investigation was performed with push test on 29 specimens. Five types with two specimens per type were prepared with HIDECK 75 steel sheeting, four types with two specimens per type were prepared with BONDEK 600 steel sheeting, and two types with three specimens per type were prepared with FR38/158 steel sheeting. Also, one type of solid slab 120 mm with two specimens and one type of solid slab 100 mm with three specimens were prepared and tested.

Data regarding description of specimens could be seen in Table 1.

3.1. HIDECK 75 Profiled Steel Sheeting

Five types with total of 10 specimens were prepared using HIDECK 75 steel sheeting ($d = 1{,}0$ mm). Two types (1.1, 1.2, 2.1 and 2.2) are with transversal position of ribs and welded through deck. The first one (1.1 and 1.2) is with two shear connectors and the second is with one shear connector in cross-section (Fig. 8).

Reducing factor k_t (4), for $b_o = 115$, $h_p = 75$, $h_{sc} = 100$ and $n_r = 2$ is 0,253 < $k_{t,\max} = 0{,}70$.

Expected failure load (2) for type 1 is 104,16 × 0,253 × 8 = 210,8 kN.

Reducing factor k_t (4), for $b_o = 115$, $h_p = 75$, $h_{sc} = 100$ and $n_r = 1$ is 0,358 < $k_{t,\max} = 0{,}85$.

Expected failure load (2) for type 2 is 104,16 × 0,358 × 4 = 149,2 kN.

The steel beam is IPE270 (S275JR) and the reinforcement is Q188 (Ø6/15 cm).

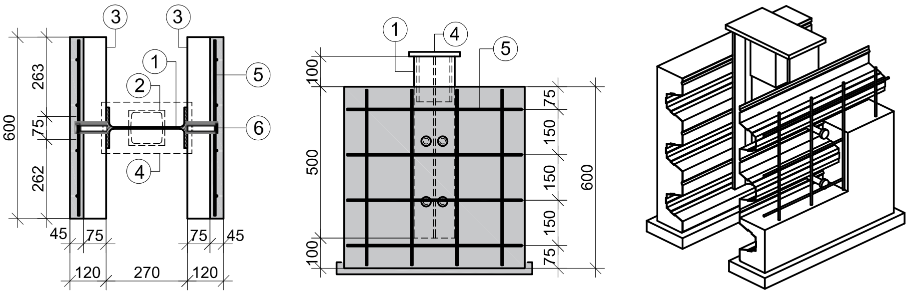

Figure 8. HIDECK 75 – transversal position

Three types were prepared with longitudinal position of ribs. The first one (3.1, 3.2) is with two shear connectors in cross-section welded through holes, the second (4.1, 4.2) with one shear connector in cross-section welded through holes, and the third (5.1, 5.2) with one shear connector in cross-section welded through deck (Fig. 9).

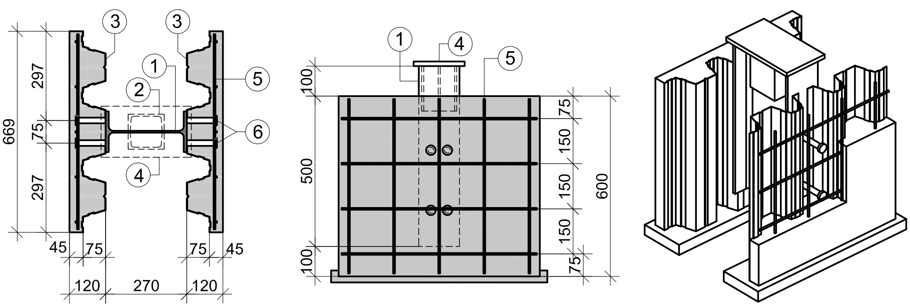

Figure 9. HIDECK 75 – longitudinal position

The steel beam is IPE270 (S275JR), and reinforcement is Q188 (Ø6/15 cm).
Reducing factor k_l (3), for $b_o = 175$, $h_p = 75$ and $h_{sc} = 100$ is 0,467.
Expected failure load (2) for type 3 is 104,16 × 0,467 × 8 = 389,1 kN.
Reducing factor k_l (3), for $b_o = 115$, $h_p = 75$ and $h_{sc} = 100$ is 0,307.
Expected failure load (2) for type 4 and 5 is 104,16 × 0,307 × 4 = 127,9 kN.

3.2. BONDECK 600 profiled steel sheeting

Four types with total of 8 specimens were prepared using BONDECK 600 steel sheeting ($d = 1{,}0$ mm). Three types (6.1, 6.2, 7.1, 7.2, 8.1 and 8.2) are with longitudinal position of ribs, two types (6 and 7) welded through holes, and one (8) welded through deck. The first one (6.1 and 6.2) is with two shear connectors, and the other two (7.1, 7.2, 8.1 and 8.2) are with one shear connector in cross-section (Fig. 10).

Reducing factor k_l (3), for $b_o = 165$, $h_p = 52$ and $h_{sc} = 100$ is 1,757 > 1, so that $k_l = 1$.
Expected failure load (2) for type 6 is 104,16 × 1 × 8 = 833,3 kN.
Expected failure load (2) for type 7 and 8 is 104,16 × 1 × 4 = 416,6 kN.
The steel beam is IPE270 (S275JR) and the reinforcement is Q188 (Ø6/15 cm).

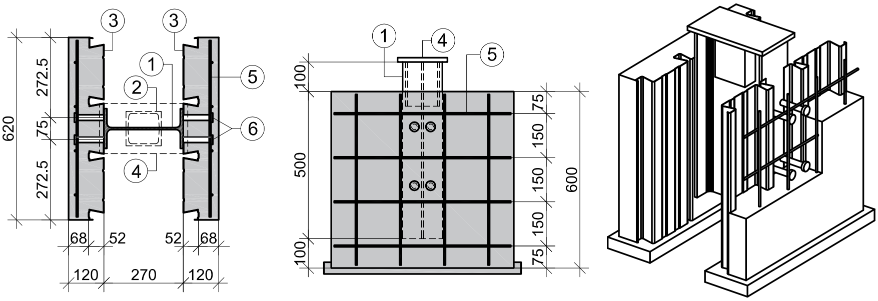

Figure 10. BONDECK 600 – longitudinal position

One type (9.1 and 9.2) was prepared with transversal position of ribs. There is one shear connector in cross-section welded through deck (Fig. 11).

Reducing factor k_t (4), for $b_o = 165$, $h_p = 52$, $h_{sc} = 100$ and $n_r = 1$ is 2,050 > $k_{t,\max} = 0{,}85$.

Expected failure load (2) for type 9 is 104,16 × 0,85 × 4 = 354,1 kN.
The steel beam is IPE270 (S275JR), and reinforcement is Q188 (Ø6/15 cm).

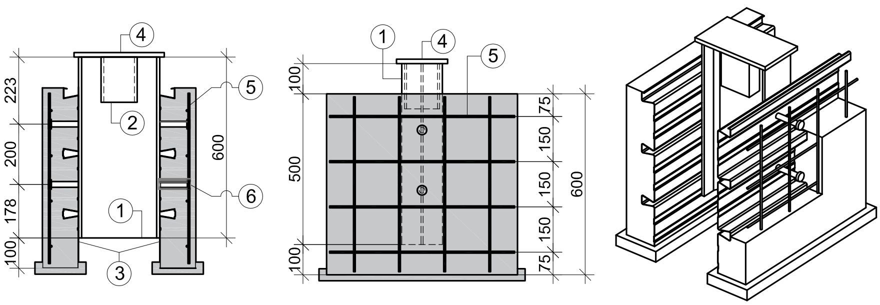

Figure 11. BONDECK 60 – transversal position

3.3. FR38/158 Profiled Steel Sheeting

FR38/158 steel sheeting is widely used in our country. This steel sheeting is without indentations or embossments, and it is not recommended for composite slabs. However, there are no limitations for the usage of this type as formwork and in composite action of steel beam and concrete slab. Experimental investigation of composite beams with concrete slab cast on FR38/158 is in progress at our laboratory.

Two types with total of 6 specimens were prepared using FR38/158 steel sheeting ($d = 1{,}0$ mm). One type (10.1, 10.2 and 10.3) is with transversal position of ribs, one shear connector in cross-section, and welded through deck (Fig. 12).

Reducing factor k_t (4), for $b_o = 79$, $h_p = 38$, $h_{sc} = 100$ and $n_r = 1$ is 2,374 > $k_{t,\max} = 0{,}85$.

Expected failure load (2) for type 10 is 104,16 × 0,85 × 6 = 531,2 kN.

The steel beam is IPE240 (S275JR) and the reinforcement is Q188 (Ø6/15 cm).

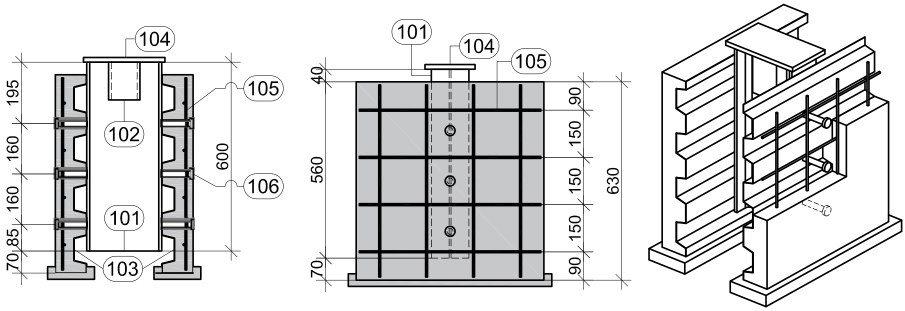

Figure 12. FR38/158 – transversal position

One type (11.1, 11.2 and 11.3) is with longitudinal position of ribs, one shear connector in cross-section and welded through holes (Fig. 13).

Reducing factor k_l (3), for $b_o = 79$, $h_p = 38$ and $h_{sc} = 100$ is 2,035 > 1, so that $k_l = 1$.

Expected failure load (2) for type 11 is 104,16 × 1 × 6 = 625,0 kN.

The steel beam is IPE240 (S275JR) and the reinforcement is Q188 (Ø6/15 cm).

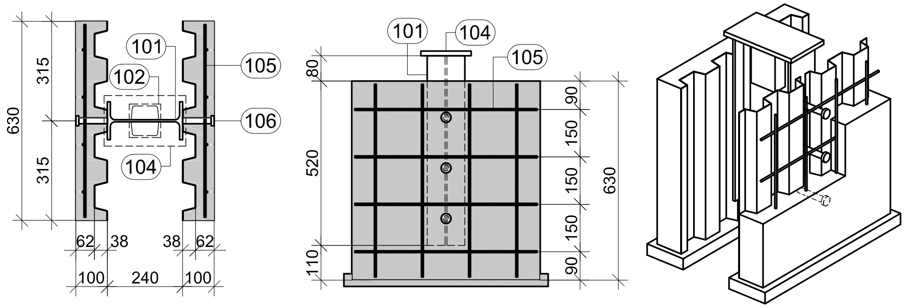

Figure 13. FR38/158 – longitudinal position

3.4. Solid Slab

In order to compare the results of load capacity, two types with solid slab were prepared. First type (12.1, 12.2) with slab depth of 12 cm, and second (13.1, 13.2, 13.3) with 10 cm (Fig. 14).

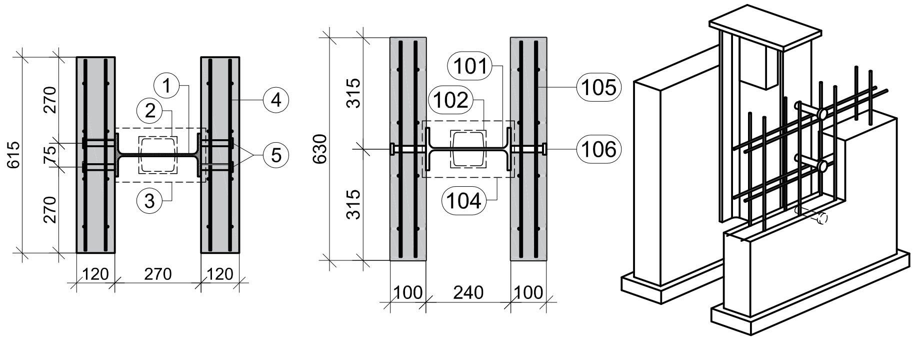

Figure 14. Solid slabs

The steel beam for type 12 is IPE270 (S275JR), for type 13 is IPE240, and for both types the reinforcement is Q188 (Ø6/15 cm).

4. Results of Testing

General description of specimens and data for measured and expected load capacity are presented in Table 1.

Table 1. Specimens characteristics, measured and expected forces

Spec. No.	Type of prof. steel sheeting	Position L long. T trans.	Number of shear connect.	Welding TD thro. deck H with holes	P_{max} exp. [kN]	P_{max} EC4 [kN]
1.1	HIDECK 75	T	2	TD	247,5	210,8
1.2	HIDECK 75	T	2	TD	247,5	210,8
2.1	HIDECK 75	T	1	TD	242,9	149,2
2.2	HIDECK 75	T	1	TD	261,9	149,2
3.1	HIDECK 75	L	2	H	X	389,1
3.2	HIDECK 75	L	2	H	761,5	389,1
4.1	HIDECK 75	L	1	H	495,4	127,9
4.2	HIDECK 75	L	1	H	384,9	127,9
5.1	HIDECK 75	L	1	TD	274,1	127,9
5.2	HIDECK 75	L	1	TD	369,2	127,9
6.1	BONDECK	L	2	H	830,6	833,3
6.2	BONDECK	L	2	H	834,4	833,3
7.1	BONDECK	L	1	H	509,1	416,6
7.2	BONDECK	L	1	H	527,0	416,6

Spec. No.	Type of prof. steel sheeting	Position L long. T trans.	Number of shear connect.	Welding TD thro. deck H with holes	P_{max} exp. [kN]	P_{max} EC4 [kN]
8.1	BONDECK	L	1	TD	482,5	416,6
8.2	BONDECK	L	1	TD	445,6	416,6
9.1	BONDECK	T	1	TD	399,0	354,1
9.2	BONDECK	T	1	TD	434,8	354,1
10.1	FR38/158	T	1	TD	462,9	531,2
10.2	FR38/158	T	1	TD	431,8	531,2
10.3	FR38/158	T	1	TD	399,9	531,2
11.1	FR38/158	L	1	H	570,3	625,0
11.2	FR38/158	L	1	H	486,5	625,0
11.3	FR38/158	L	1	H	520,3	625,0
12.1	Solid slab 12		2		944,5	833,3
12.2	Solid slab 12		2		839,7	833,3
13.1	Solid slab 10		1		511,6	625,0
13.2	Solid slab 10		1		554,0	625,0
13.3	Solid slab 10		1		571,6	625,0

In the following graphics the measured maximum force for each specimen and P-δ behavior are presented. The horizontal line shows the expected maximum force. The ductile behavior of the shear connector, according to EC4, requests at least 6 mm slip capacity in push test at characteristic load level ($0{,}9P_{max}$).

The measured maximum forces for type 1 (Fig. 15) are relatively close to EC4 analytical prediction. The request for ductility is not satisfied. Failure occurred in concrete.

The measured maximum forces for type 2 (Fig. 16) are significantly higher than EC4 analytical prediction. The request for ductility is not satisfied. Failure occurred in concrete.

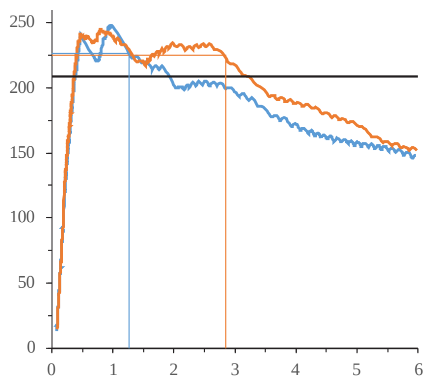

Figure 15. Specimens 1.1 and 1.2

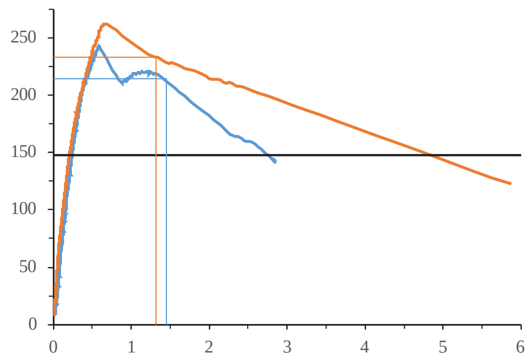

Figure 16. Specimens 2.1 and 2.2

The measured maximum force for type 3 (Fig. 17) is significantly higher (nearly 100%) than EC4 analytical prediction. The request for ductility is almost satisfied. Failure occurred in concrete.

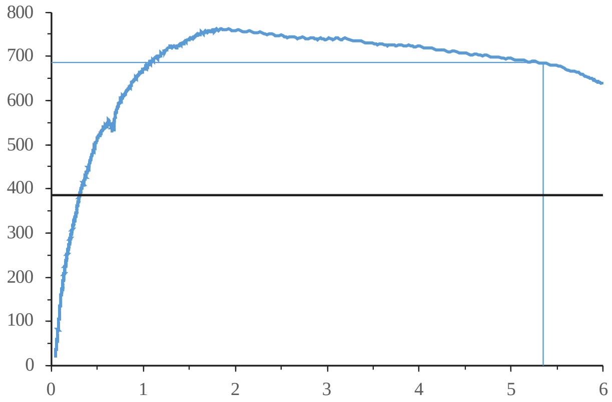

Figure 17. Specimens 3.2

The measured maximum forces for type 4 (Fig. 18, upside curves) are significantly higher (3,5 times) than EC4 analytical prediction. The request for ductility is not satisfied. Failure occurred in concrete.

Also, the measured maximum forces for type 5 (Fig. 18, downside curves), are significantly higher (2,5 times) than EC4 analytical prediction. The request for ductility is satisfied. Failure occurred in concrete.

The measured maximum forces in all specimens prepared with HIDECK 75, especially the cases with longitudinal position of ribs, are significantly higher than EC4 analytical prediction. For steel sheeting with higher ribs (h_p), there is evident underestimation of headed stud shear resistance proposed by EC4. The values of reduction factors (k_l, k_t) mostly depend on the value of rib height (h_p). The entire height of the concrete, or the height of the concrete above ribs, is not taken into consideration.

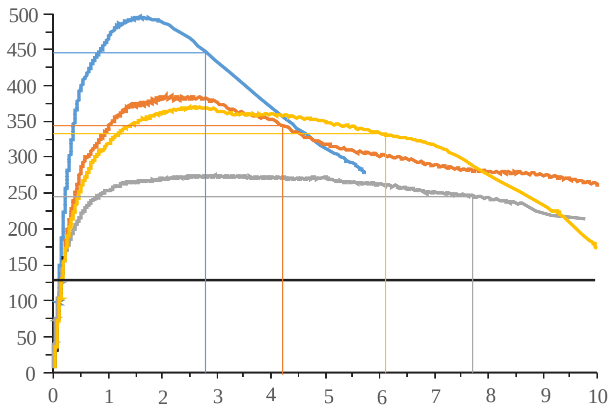

Figure 18. Specimens 4.1, 4.2, 5.1 and 5.2

The measured maximum forces for type 6 (Fig. 19) are in very good accordance with EC4 analytical prediction. The request for ductility is satisfied. Failure occurred in concrete.

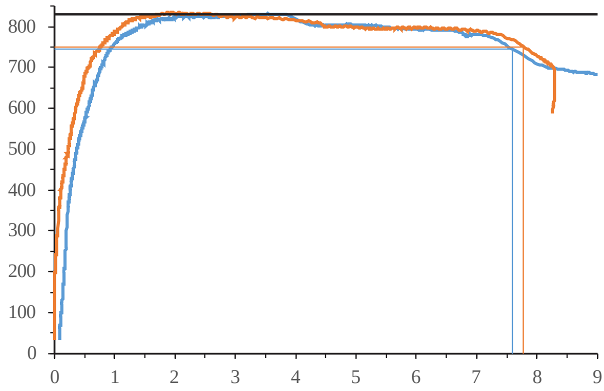

Figure 19. Specimens 6.1 and 6.2

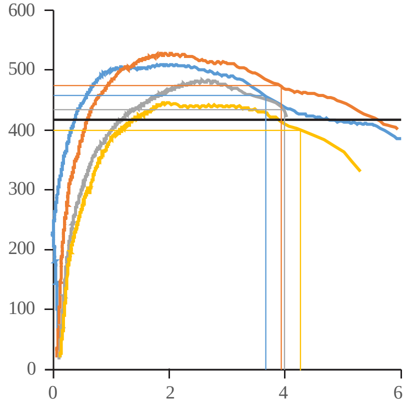

Figure 20. Specimens 7.1, 7.2, 8.1 and 8.2

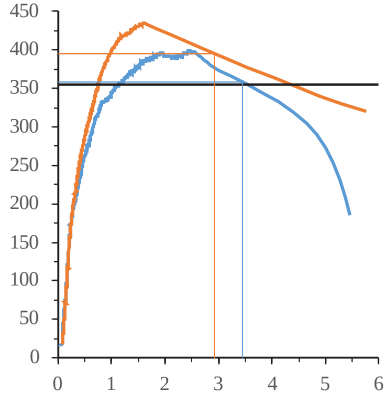

Figure 21. Specimens 9.1 and 9.2

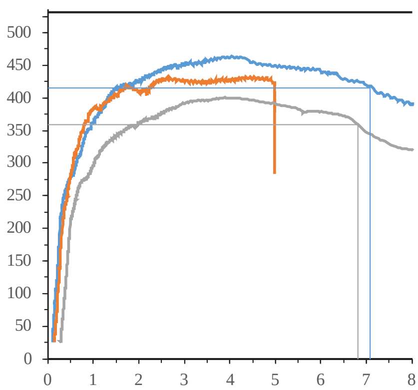

Figure 22. Specimens 10.1, 10.2 and 10.3

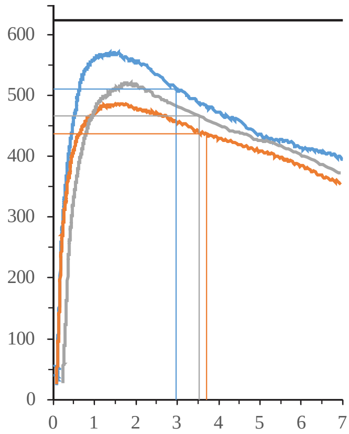

Figure 23. Specimens 11.1, 11.2 and 11.3

The measured maximum forces for type 7 (Fig. 20, upside curves) are relatively close to EC4 analytical prediction. The request for ductility is not satisfied. Failure occurred in concrete.

The measured maximum forces for type 8 (Fig. 20, downside curves) are relatively close to EC4 analytical prediction. The request for ductility is not satisfied. Failure by shear of connectors occurred on specimen 8.1 and failure in concrete occurred on specimen 8.2.

The measured maximum forces for type 9 (Fig. 21) are relatively close to EC4 analytical prediction. The request for ductility is not satisfied. Failure by shear of connectors occurred on specimen 9.1 and failure in concrete occurred on specimen 9.2.

The measured maximum forces in all specimens prepared with BONDECK 600, steel sheeting with re-entrant profile are in good accordance with EC4 analytical prediction.

The measured maximum forces for type 10 (Fig. 22) are approximately 19% lower than EC4 analytical prediction. Specimens 10.1 and 10.3 satisfied the request for ductility, and failure occurred in concrete. Failure by shear of connectors occurred on specimen 10.2.

The measured maximum forces for type 11 (Fig. 23) are approximately 16% lower than EC4 analytical prediction. The request for ductility is not satisfied. Failure occurred in concrete.

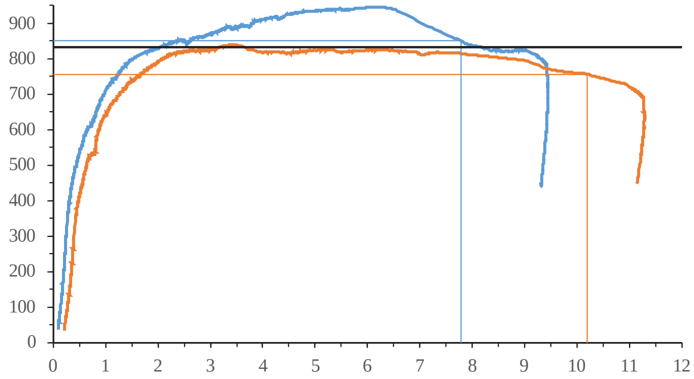

Figure 24. Specimens 12.1 and 12.2

The measured maximum forces for type 12 (Fig. 24) are very close with EC4 analytical prediction. The request for ductility is satisfied. Failure occurred in concrete.

The measured maximum forces for type 13 (Fig. 25) are approximately 13% lower than EC4 analytical prediction. The request for ductility is satisfied. Failure occurred in concrete.

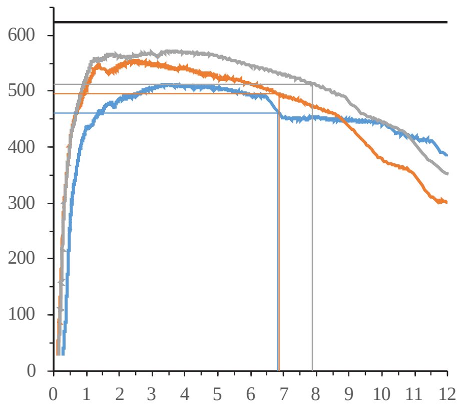

Figure 25. Specimens 13.1, 13.2 and 13.3

The following pictures illustrate the failure state of some specimens.

Figure 26. Specimen 1.2

Figure 27. Specimen 6.2

Figure 28. Specimen 11.1

Figure 29. Specimen 13.2

5. Conclusions

From the experimental investigation of the load and slip capacity of headed stud shear connectors in composite slabs for buildings presented in this paper might be concluded:

There is evident underestimation of headed stud shear resistance proposed by EC4 in the case of steel sheeting with higher ribs (HIDECK 75). The entire height of the concrete, or the height of concrete above ribs, should be taken in expressions for reduction factors (k_l, k_t).

For re-entrant form steel sheeting (BONDECK 600) there is good accordance of headed stud shear resistance with EC4 analytical prediction.

In the case of FR38/158 steel sheeting, open form shape, widely used as formwork for floor slabs in our country, measured shear resistance of headed studs, with no concrete cover, is about 20% below EC4 analytical prediction.

In solid slab (12 cm) with 2 cm concrete cover, there is very good accordance of headed stud shear resistance with EC4 analytical prediction.

In solid slab (10 cm) with no concrete cover, measured headed stud shear resistance is about 15% below EC4 analytical prediction.

The shear resistance of through deck welded headed studs, for longitudinal position of steel sheeting, is higher than those welded through holes. There is no additional correction, or limitation, for reduction factor k_l in EC4 regarding the method of welding.

Headed studs in solid slabs fulfil the request for ductility according to EC4. The request for ductility for types with steel sheeting is satisfied occasionally.

For steel sheeting with low ratio h_c / h_p, additional experimental investigations are recommended.

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