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Effect of local small diameter stud connectors on behavior of partially encased composite beams

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Abstract. The paper combines two distinct parts. First the behavior of welded headed studs with small diameters of 10 and 13 mm acting as shear connectors (which are not embraced in current standards) is studied. Based on standard push tests the load-slip relationships and strengths are evaluated. While the current standard (Eurocode 4 and AISC) formulas used for such studs give reasonable but too conservative strengths, less conservative and full load-slip rigidities are evaluated and recommended for a subsequent investigation or design. In the second part of the paper the partially encased beams under bending are analyzed. Following former experiments showing rather indistinct role of studs used for shear connection in such beams their role is studied. Numerical model employing ANSYS software is presented and validated using former experimental data. Subsequent parametric studies investigate the longitudinal shear between steel and concrete parts of the beams with respect to friction at the steel and concrete interface and contribution of studs with small diameters required predominantly for assembly stages (concreting). Substantial influence of the friction and effect of concrete confinement was observed with rather less noticeable contribution of the studs. Distribution of the longitudinal shear and its sharing between friction and studs is presented with concluding remarks.

Keywords: composite beams; nonlinear behavior; numerical modeling; partially encased beams; push-out tests; shear transfer; small studs

1. Introduction

Composite steel and concrete beams and columns are considered as a reasonable choice attaining proper balance between advantages they offer and a respective cost. The basic variants of steel and concrete composite elements comprise steel beams with a concrete slab, partially encased steel elements, fully encased steel elements and concrete-filled steel hollow sections. Full encasement is usually expensive in relation to the benefits obtained and therefore the use of the partial rather than the full encasement of steel profiles have proved to be more popular in Europe in recent years. Partially-encased members, where concrete is placed between steel member flanges only, result in several advantages, such as the high fire-resistance and higher load capacity, as well as significant improvements with respect to buckling of steel parts, increase of bending

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stiffness and a reduction of deflections compared to a sole steel beam. Other practical advantages include the use of conventional steel connections details as well as a reduction or omission of otherwise necessary formwork.

Basic recommendations for design of such sections are given in EN 1994-1-1 (2004), in the following termed as Eurocode 4. The concrete that encases a web and flanges shall be longitudinally reinforced and mechanically connected to the steel section by welding stirrups or by bars going through web holes or by studs with a diameter $d \geq 10$ mm at spacing ≤ 400 mm. Research concerning more detailed behavior of partially encased sections embodies large spectrum of questions. Elnashai *et al*. (1991) studied experimentally beam-columns with conventional and modified cross-sections (with stirrups and additional welded bars - links) under cyclic and dynamics loading and referred to improvement of ductility and local flange buckling due to the modification. Kindmann *et al*. (1993) tested 12 beams with partially encased HE400AA cross-sections and various steel-concrete bond measures. They pointed out to a negligible slip between steel and concrete parts in all specimens and suggested to take the encased concrete and reinforcement into account for both bearing capacity and rigidity. Another 8 partially encased beams tested Assi *et al*. (2002) using lightweight and normal concrete and results compared with behavior of bare steel sections. While substantial contribution to strength and rigidity due to encasement was observed, the difference of the both, i.e., between normal and lightweight concrete, was found to be very small. Detailed numerical analysis of partially encased columns was presented by Chicoine *et al*. (2002). They studied former column tests of sections with transverse links attached to the flanges using ABAQUS software, including nonlinear behavior, residual stresses and initial imperfections. The study recommends a design equation for determining the axial capacity of such columns. Combined effects of headed studs, reinforcing bars and friction at steel-concrete interface were studied by Hegger and Goralski (2005). Absence of the headed studs or reinforcement led to less ductile behavior and different failure modes. Local buckling of thin flanges of partially encased composite columns was numerically studied by Begum *et al*. (2007). Using dynamic explicit formulation in nonlinear FEM including local imperfections, the strength capacity was determined and discussed with experimental results. Numerical model of partially encased beam without encasement reinforcement was developed by Piloto *et al*. (2006). Using ANSYS software they studied lateral torsional buckling with simplified nonlinear material behavior and replaced bond between steel and concrete by nonlinear springs with defined load-slip diagram. In comparison with simple steel beams the buckling capacity of the partially encased beams increased for small relative slenderness' up to 0.6, while the influence of encasement for higher slenderness' proved to be insignificant. Hanna and Amin (2006) investigated numerically capacity of partially encased columns with welded bars (links) under axial loading, with good agreement with former tests by Tremblay (Tremblay *et al*. 2000) and Chicoine (Chicoine *et al*. 2002).

Recently, several investigations focused primarily on the inelastic performance of partially-encased members with headed studs welded on the web under combined bending and axial loading. Elghazouli and Treadway (2008) tested 10 samples with various HEA profiles of Grade S460 steel both under major- and minor-axis bending. Evaluation of tests covered hardening of steel, confinement effects of concrete and buckling of the profiles flanges. Proposals for the determination of moment capacities and increase of stiffness due to a concrete contribution were given. De Nardin and El Debs (2009) tested three beams of partially encased asymmetrical I cross sections. Two positions of 19 mm headed studs as shear connectors were investigated, welded either to web or bottom flange, while no other concrete reinforcement was used. The tests indicated little effect of the studs in whichever position on ultimate capacity in comparison with beams without using any studs as shear connectors. Nevertheless, ductile behavior due to the studs was demonstrated and placement of the studs recommended. Shukur (2009) analyzed the former tests by nonlinear analysis using ANSYS software. Model of stud shear connection consisted of nonlinear springs and a friction between steel and concrete by using isotropic Coulomb friction. Shukur found the strains at steel-concrete interface nearly unaffected by presence of stud shear connectors. Six tests were performed by Chen *et al*. (2010), in which three types of constructional detailing was studied under combined axial and transverse cyclic loading. The encased concrete was equipped either with transverse links (bars) only, or additional reinforcement or even steel stirrups. ANSYS FE simplified model was developed using nonlinear springs for concrete, reinforcement and the links. The three detailing types showed negligible differences in behavior. Another tests of partially encased beams with steel parts of Class 4 cross-sections were studied by Kvocak and Drab (2012) but little details were supplied. Begum and Ghosh (2014) presented an attempt to simulate numerically behavior of a partially encased column by an equivalent steel section.

Concerning welded headed stud shear connectors the basic recommendations are given in the Eurocode 4. Design strength and ductility conditions resulted from a number of push-out tests, which are normalized by the Eurocode 4. Formulas are valid for studs with diameter $16 \leq d \leq 25$ [mm], while load-slip relationships provides a rich source of references, e.g., Ollgaard *et al*. (1971), Johnson and Oehlers (1981), Oehlers and Coughlan (1986). Large studs with diameter up to 30 mm were investigated by Shim (2004), resulting in static and fatigue strengths and their comparison with Eurocode formulas. Numerical modeling of headed studs by means of horizontal and vertical springs was verified by Wang and Chung (2006). Mirza and Uy (2009) analyzed headed studs under interaction of axial and shear loading. Using ABAQUS software they developed 3D nonlinear finite element model and results validated on a range of push-out tests. Influence of axial tension load with respect to shear stud capacity resulted into interaction diagram and arrangement of concrete slab thickness and reinforcement was studied. Later Mirza and Uy (2010) extended the research for influence of time-dependent behavior of concrete. Behavior of single and doubled welded studs in a light concrete was investigated by Valente and Cruz (2009). Other arrangements of welded headed studs include “lying” shear studs, where the behavior is influenced by small distance from concrete slab edge (Kuhlmann and Kürschner 2005) and studs in a group arrangement (Okada *et al*. 2006).

During last decades the use of welded headed studs proved to be simple and efficient. Nevertheless, new technologies, arrangements and computer techniques enable to widen knowledge and novel use of these connectors.

In the first part of this paper are described tests with small diameter welded headed studs. Studs with diameters 10 and 13 [mm] were investigated in standard push tests to receive full load-deflection relationships for determination of their strength and ductility not covered by Eurocode 4. In the second part of the paper the behavior of partially encased beams, in which longitudinal shear was transferred by small diameter studs and through friction in the steel-concrete interface, is studied numerically. Validation of ANSYS software nonlinear modeling was performed using experiments by Kindmann and Bergmann (Kindmann *et al*. 1993). As the friction proved to be deciding, parametric studies with various friction coefficients were performed to show their significance.

Fig. 1 Welding source LBH 910 and welding gun PHM – 161

Table 1 Test program

Series	Stud diameter d [mm]	Specimen	Concrete grade	Series	Stud diameter d [mm]	Specimen	Concrete grade
T1	10	T1S1	C20/25	T4	10	T4S1	C30/37
		T1S2	C20/25			T4S2	C30/37
		T1S3	C20/25			T4S3	C30/37
T2	13	T2S1	C20/25	T3	13	T3S1	C30/37
		T2S2	C20/25			T3S2	C30/37
		T2S3	C20/25			T3S3	C30/37

2. Headed studs with small diameters

Behavior of headed studs with diameters of d = 10 and 13 mm was investigated experimentally using standard push tests. The small studs not only provide a very economical shear connector but also enable easy and practical application of composite construction in small sites. Eurocode 4 specifies shear resistance of automatically welded headed stud shear connectors having diameter of 16÷25 mm. Welding of these diameters usually requires special equipments at site, particularly with respect to welding current (around 1800 A) and current protection (63 A). Present-day devices for welding with stroke ignition (arc) are signalized with LED diode and welding time on a display, have adjustable welding time, constant welding current and parameters are optimized for a given range of stud diameters including the smaller ones than assumed by Eurocode. For example, source LBH 910 can be used for headed studs up to diameter of 14 mm, while the welding current is around 1000 A, voltage 400/50 and current protection 32 A only. The appropriate welding gun PHM-161 with a welding time 5-900 ms uses the advanced mechanism for alignment of length tolerances and automatic adjustment of stroke, Fig. 1. In total 12 push-out tests was performed with denomination according to Table 1.

2.1 Stud welding, test specimens and testing

Headed studs of type SD (KB-Kopfbolzen) which are generally available were used, made of steel S235J2G3+C450 according to EN ISO 13918 (DIN 32500 Teil 3), with nominal tensile strength f_u = 450 MPa. Before using these studs, their material properties were determined from 5 standard tensile tests for each stud diameter. The yield stress was determined as 0.2% proof stress, because the steel is thermally treated and does not show a marked yielding flow. The average

Fig. 2 Standard stud bent test and steel part of specimens with studs

Table 2 Characteristics of concrete

Series	Concrete grade	Number of samples	Time of testing after concreting	Compressive strength [MPa]		Elastic modulus $E_{c,exp}$ [MPa]
				σ_{cube}	$\sigma_{c,exp}$	
T1	C20/25	3	32	26.7	21.4	29510
T2	C20/25	3	32	32.6	26.1	31057
T3	C30/37	3	32	39.1	31.3	34600
T4	C30/37	3	32	35.5	28.4	31500

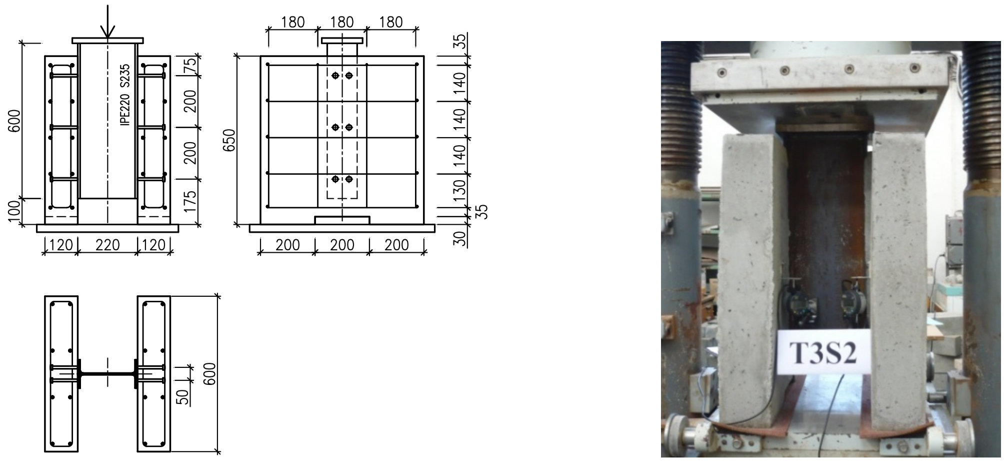

Fig. 3 Test specimen and testing

ultimate strength and elongation at rupture were $f_{u,exp}$ = 547.3 MPa and δ_5 = 13.4% for the 10 mm studs while 495.2 MPa and 17.0% for the 13 mm studs. Welding of the studs was carried out with source LBH 1400 and gun PHM-161 while the quality control of the welding process was verified by 45° bent proof as prescribed for large studs, Fig. 2.

Concrete slabs were produced from two prescribed compressive strength of concrete (nominally C20/25 and C30/37). For each mix, samples of concrete were prepared at the time of casting, to determine the concrete strength and elastic modules at time of the push-out testing. The compressive strength was obtained from testing of cube samples, while mean cylinder strength

$\sigma_{c,exp}$ was derived as 80% of the former. Table 2 summarizes the mean material properties of used concretes.

Test specimens were fabricated according to Eurocode 4, appendix B. Concrete slabs were of size 600×550×120 [mm], concreted successively one after another, had prescribed reinforcement Ø 10 mm of steel B500 B and the interface between the flanges of steel section and the concrete slab was prevented by greasing the flanges. The specimens had six connectors arranged in two rows and embedded in each slab, as illustrated in Figs. 2 and 3.

Push-out specimens were tested in a hydraulic testing machine EDB 400U, Fig. 3. The load was applied in increments up to 40% of the expected failure load and then cycled 25 times between 5% and 40% of the expected failure load. Subsequent load increments were imposed in the way that failure did not occur in less than 15 minutes. Longitudinal slips between each concrete slab and the steel section were measured continuously during loading or at each load increment.

2.2 Test results

Load-slip curves for each individual series are shown in Figs. 4 and 5. The studs failed due to a shear of shank base or a concrete bearing followed by collapse, Fig. 6.

Resulting average *P*-*δ* relationships were received from digital data (e.g., see Table 3 for T1 Series). Approximation of the elastic slips and elastic shear stiffnesses of all series is given in Table 4.

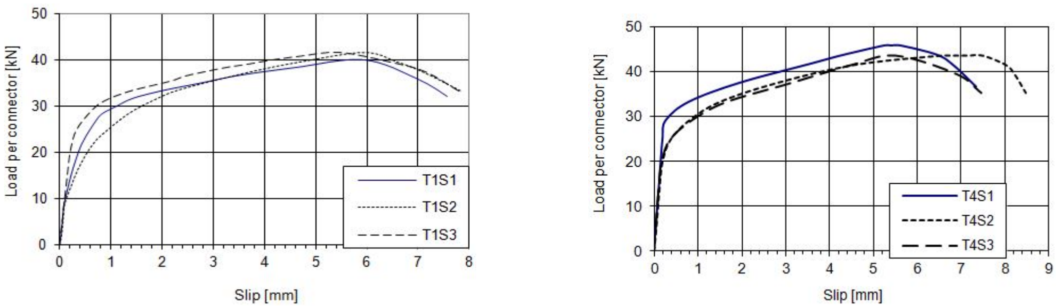

Fig. 4 Load-slip relationship of studs with diameter 10 mm (Series T1 and T4)

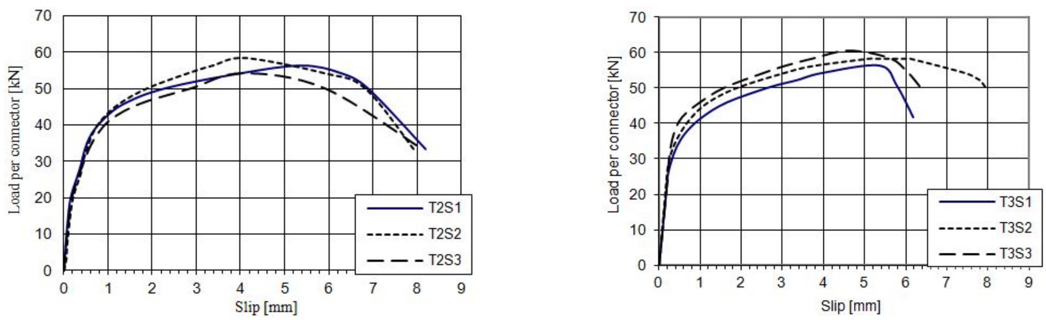

Fig. 5 Load-slip relationship of studs with diameter 13 mm (Series T2 and T3)

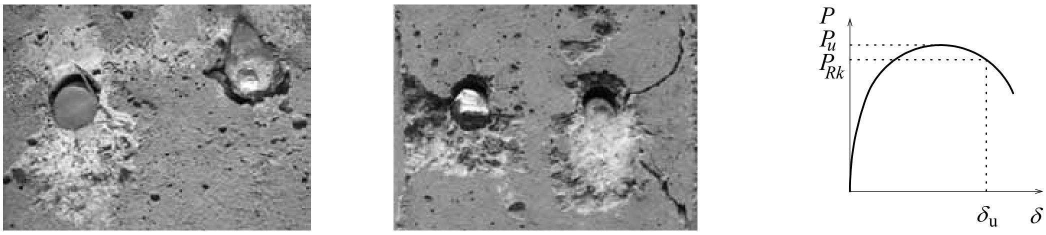

Fig. 6 Typical stud failure traces and evaluation of stud slip capacity

Table 3 Shear force-slip relationship for test T1 (10 mm diameter studs)

P/P_{max} [%]	P [kN]	δ [mm]	P/P_{max} [%]	P [kN]	δ [mm]
0	0	0.00	60	25.00	0.630
5	2.08	0.029	70	29.17	1.022
16	6.67	0.072	75	31.25	1.355
20	8.33	0.086	80	33.33	1.885
25	10.42	0.122	85	35.42	2.687
30	12.50	0.172	92	38.33	3.979
35	14.58	0.219	94	39.17	4.488
40	16.67	0.270	96	40.00	5.003
50	20.83	0.404	100	41.11	5.707
60	25.00	0.630	90	37.50	7.27

Table 4 Average elastic slip and stiffness

Series	Stud diameter d [mm]	$\sigma_{c,exp}$ [MPa]	Elastic slip [mm]	Elastic stiffness [kN/mm]
T1	10	21.4	0.173	81.0
T2	13	26.1	0.177	114.9
T3	13	31.3	0.238	122.9
T4	10	28.4	0.222	105.9

2.3 Design values of the small diameter studs

Because of limited number of specimens the evaluation of characteristic resistance P_{Rk} and characteristic ultimate slip δ_{uk} was not performed statistically but in a simplified way according to Eurocode 4 (i.e., the characteristic resistance $P_{Rk,exp}$ as the minimum failure load $P_{u,exp}$ reduced by 10% and corresponding characteristic slip capacity δ_{uk} as the minimum test value $\delta_{u,exp}$ reduced by 10%, see Fig. 6.) and are given in Table 5.

According to Eurocode 4 the 10 mm studs should be considered as ductile ($\delta_{uk} \geq 6$ mm) while 13 mm studs as non-ductile. However, the fracture similarities and nearness to the required value of 6 mm enables to consider in a design both as ductile.

Eurocode 4 specifies two expressions for characteristic strength of studs with diameters d = 16÷25 mm valid in case of long studs ($h_{sc} > 4\ d$)

$$P_{Rk,EN1} = 0.8\, f_u \frac{\pi\, d^2}{4}; \qquad P_{Rk,EN2} = 0.29\, d^2 \sqrt{f_{ck} E_{cm}} \tag{1}$$

AISC Specifications for Structural Steel Buildings (2010) use similar formula (here rewritten using Eurocode 4 symbols)

$$P_{Rk,AISC} = 0.39\, d^2 \sqrt{f_{ck} E_{cm}} \le 0.75\, f_u \frac{\pi\, d^2}{4} \tag{2}$$

Comparison of the all 12 test results with strengths resulting from the above formulas in which measured values of $f_u = f_{u,exp}$, $f_{ck} = \sigma_{c,exp}$ and $E_{cm} = E_{c,exp}$ were used as given in previous Chapters are presented in Table 6 and shown in Fig. 7.

Consequently, based on the limited amount of tests, the approximate and very conservative characteristic resistances P_{Rk} of the given small studs SD (KB-Kopfbolzen) for common nominal qualities of concrete C20/25 up to C30/37 may be taken from the Eurocode 4 formulas according to Eq. (1), but more appropriately from AISC formula Eq. (2). Nevertheless, procedure given in Eurocode 4 for evaluation of tests gives much higher and sufficiently conservative values $P_{Rk,exp}$ (see the last column in Table 6 and Fig. 7). Partial safety factor for determination of design resistance may be taken as $\gamma_v = 1.25$ in accordance with Eurocode 4.

Table 5 Test results

Series	Specimen	Strength per one stud [kN]		Slip [mm]	
		$P_{u,exp}$	$P_{Rk,exp}$	$\delta_{u,exp}$	δ_{uk}
T1	T1S1	40.00	36.0	7.16	6.4
	T1S2	41.67		7.20	
	T1S3	41.67		7.45	
T2	T2S1	56.25	48.8	6.88	5.5
	T2S2	58.33		6.27	
	T2S3	54.17		6.09	
T3	T3S1	56.25	50.6	6.11	5.5
	T3S2	58.33		7.78	
	T3S3	60.42		6.80	
T4	T4S1	45.83	39.4	6.84	6.2
	T4S2	43.75		8.23	
	T4S3	43.75		6.90	

Table 6 Comparison of resistances

Series	Specimen	d [mm]	$f_{u,exp}$ [MPa]	$\sigma_{c,exp}$ [MPa]	$E_{c,exp}$ [MPa]	$P_{Rk,EN1}$ [kN]	$P_{Rk,EN2}$ [kN]	$P_{Rk,AISC}$ [kN]	$P_{u,exp}$ [kN]	$P_{Rk,exp}$ [kN]
T1	T1S1	10	547.3	21.4	29510	34.4	23.0	31.0	40.00	36.0
	T1S2								41.67	
	T1S3								41.67	

Table 6 Comparison of resistances

Series	Specimen	d [mm]	$f_{u,exp}$ [MPa]	$\sigma_{c,exp}$ [MPa]	$E_{c,exp}$ [MPa]	$P_{Rk,EN1}$ [kN]	$P_{Rk,EN2}$ [kN]	$P_{Rk,AISC}$ [kN]	$P_{u,exp}$ [kN]	$P_{Rk,exp}$ [kN]
T2	T2S1	13	495.2	26.1	31057	52.6	44.1	49.3	56.25	48.8
	T2S2								58.33	
	T2S3								54.17	
T3	T3S1	13	495.2	31.3	34600	52.6	51.0	49.3	56.25	50.6
	T3S2								58.33	
	T3S3								60.42	
T4	T4S1	10	547.3	28.4	31500	34.4	27.4	32.2	45.83	39.4
	T4S2								43.75	
	T4S3								43.75	

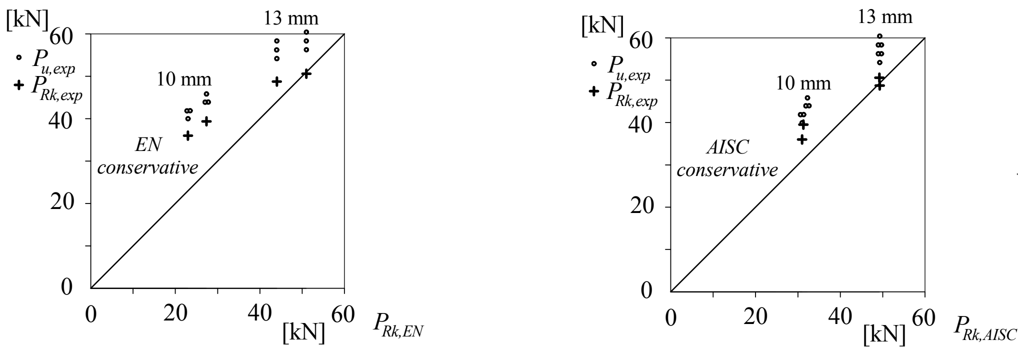

Fig. 7 Comparison of experimental strengths with expressions according to EN 1994-1-1 (2004) and AISC (2010)

3. Partially encased beams with studs of small diameters

Tests with partially encased beams indicate rather small influence of studs welded to the beam web, but in contrast draw attention to friction between steel and concrete parts and confinement effects, see references given in Chapter 1. Nevertheless, the mechanical connectors are important during the course of concreting when the beam is turned upside down and to prevent web buckling. In the following the effect of small diameter studs is investigated numerically and the model is validated using tests published by Kindmann *et al*. (1993).

3.1 Modeling and numerical analysis

The studied beam is shown in Fig. 8 and described with details in Chapter 3.3. The ANSYS software was used to model in 3D the partially encased composite steel and concrete girder with headed studs welded to its web. The materially nonlinear approach considering both headed studs and a friction along the steel and concrete interface to carry longitudinal shear was employed.

Due to symmetry only half span of the beam was analyzed, using mapped mesh method to find

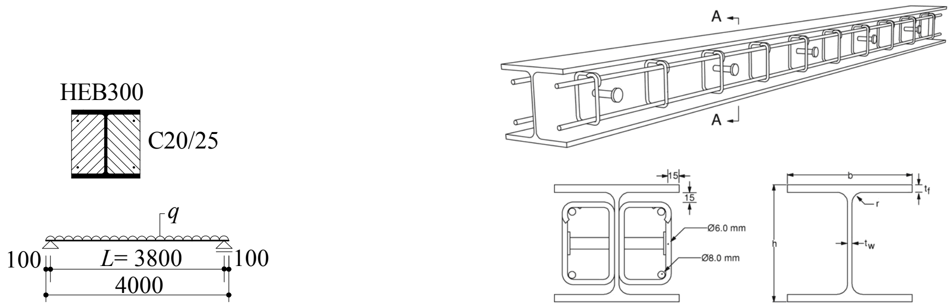

Fig. 8 Studied beam under uniform loading and its steel part

the satisfactory mesh size by sensitivity analysis (finally 40 longitudinal segments for concrete and 20 longitudinal segments for steel along half span). Steel beam was modeled using SOLID95 3D element (20 nodes, embodies plasticity, stress stiffening, large deflections, large strains) with bilinear isotropic hardening and corresponding to Eurocode 3, Fig. 9. Concrete encasement was modeled using SOLID65 3D reinforced concrete element (8 nodes, embodies cracking in tension, crushing in compression, shear transfer with smooth/rough cracks, plastic deformation, creep) with multilinear isotropic hardening (however, to help numerical stability, the endless one) corresponding to Eurocode 2, Fig. 9. For shear transfer the coefficients of the concrete element were considered as C1 = 0.2 for opened cracks and C2 = 0.6 for closed cracks. The steel and concrete Poisson's ratios were taken as 0.3 and 0.2, respectively. If reinforcing bars were necessary to cover, these were modeled within the SOLID65 as a dispersed reinforcement.

Surface to surface contact elements (representing contact and sliding between 3D target surfaces TARGE170 and a deformable surface of the former elements and defined by CONTA173) were used to model flexible-flexible contact between the steel-concrete interfaces, Fig. 10. Penetration of a given element with target segment elements gives contact, while in the process an isotropic Coulomb friction was introduced.

Local effects of headed studs were introduced by nonlinear springs (COMBIN39) to represent the shear connectors located uniformly in distance of 400 mm along span. The nonlinear spring element enables any nonlinear relation between force and extension to model correctly the shear force in the longitudinal direction (Machacek and Cudejko 2009). In case of the small diameter

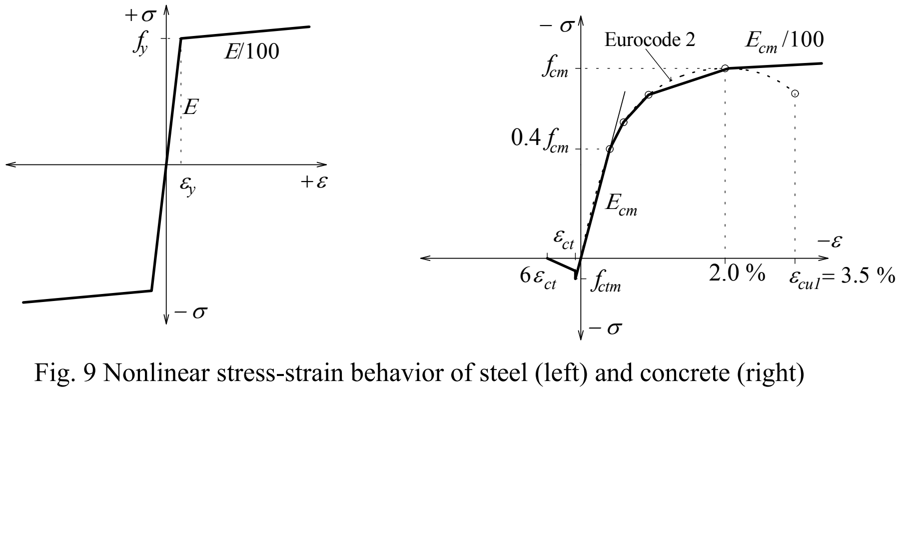

Fig. 9 Nonlinear stress-strain behavior of steel (left) and concrete (right)

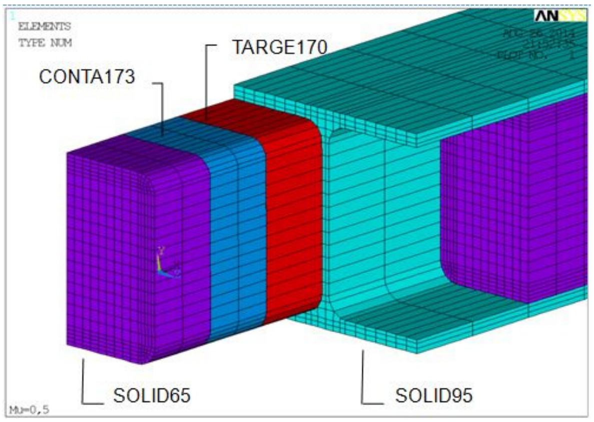

Fig. 10 FE meshing

headed studs the mean load-slip curves obtained from the above push-out tests (e.g., from Table 3) were used as the input data for the nonlinear spring behavior.

For surface-to-surface contact elements, contact algorithm "augmented Lagrangians" with default values as recommended by ANSYS was used (normal stiffness factor FKN = 0.1) and updated at each iteration step based on the current mean stress of the underlying elements and the allowable penetration (FTOLN = 0.1).

Numerical solution was performed by Newton-Raphson method using very small loading increments (10^3÷10^4 steps) to comply with the concrete nonlinearity both in compression and tension, while limiting amount of iterations (80). Collapse was always determined by a convergence failure (given by balance factors for forces 0.5% and deflections 5%) even under very small load increase, finally due to concrete malfunction (particularly in location of springs, spots of stress singularities).

3.2 Validation of the model

The model was validated against tests by Kindman *et al*. (1993). Eight samples of single composite beams and two composite beams with a concrete deck were tested. The single composite beams were equipped with various bond measures (nothing current, nothing with steel surface oiled, studs Ø 19 mm and hooks) and various longitudinal reinforcements. The results proved a substantial effect of the encased concrete for all these measures (including oiled surfaces of steel part). For simplicity the sample V8 was modeled herein, which consists of H340AA steel profile (yield f_y = 442 MPa), encased concrete (cylinder strength f_c = 45.2 MPa) and natural friction (no mechanical bonding).

The numerical model described above was employed with the friction coefficient μ = 0.3. The comparison of numerical and test results together with longitudinal stresses in steel beam are shown in Fig. 11 (note that the plasticity in steel beam was reached). Ratio of collapse loadings gives F_{num}/F_{exp} = 785/839 = 0.94 and ratio of central span deflections $\delta_{num}/\delta_{exp}$ = 12.7/11.7 = 1.08. The end slip ratio at service loading s_{num}/s_{exp} = 1.17/0.12 = 9.5 (which means that the real friction and confinement effects are much stronger) while slip presented by Kindman *et al*. and based on a calculation with no bond between steel beam and concrete was s_{cal} = 1.49 mm.

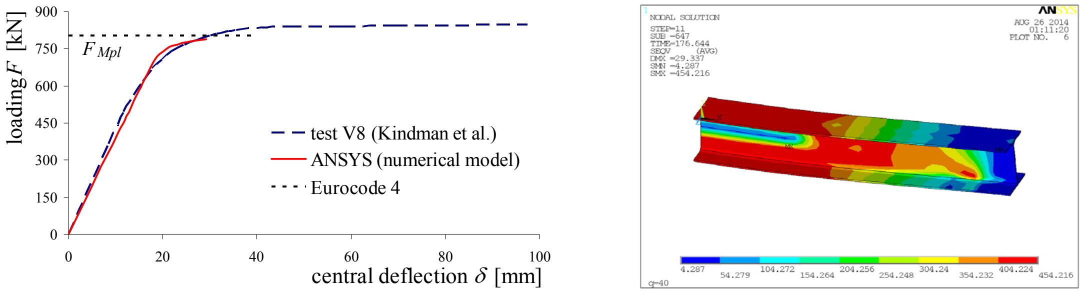

Fig. 11 Load-central deflection relationship (left) and stresses in steel beam (right)

With such results in comparison the model and entry data may certainly be improved, nevertheless, it seems to be qualified to assess the influence of a shear friction in combination with mechanical shear connectors in a parametric study.

3.3 Parametric study

The study deals with the composite beam according to Fig. 8. The steel part is of uniform rolled beam HEB300 and Grade S235 steel (f_y = 235 MPa, f_u = 360 MPa, E = 210 GPa, ν = 0.3), encased concrete is of Grade C20/25 (f_{cm} = 28 MPa, f_{ctm} = 2.2 MPa, E_{cm} = 30 GPa, ν = 0.2) and longitudinal reinforcing bars of 8 mm diameter (f_s = 500 MPa) were covered by a dispersed reinforcement within concrete finite elements. Stress-strain relationships follow Fig. 9.

The principal investigation was concentrated on influence of the steel-concrete friction and combination of friction and local headed studs with diameter 10 mm placed in the middle of the beam web at longitudinal spacing of 400 mm (P_{Rk} = 29.6 kN, see Chapter 2.3).

Load-central deflection curves for friction coefficient μ = 0.2, 0.3, 04, 0.5 and with studs is shown in Fig. 12 (for comparison also behavior of the beam without headed studs is presented). Approximate values corresponding to elastic ($q_{el,EN}$) and full plastic strengths ($q_{pl,EN}$) based on simple Eurocode 4 calculation approach with partial shear interaction (with the degree of shear

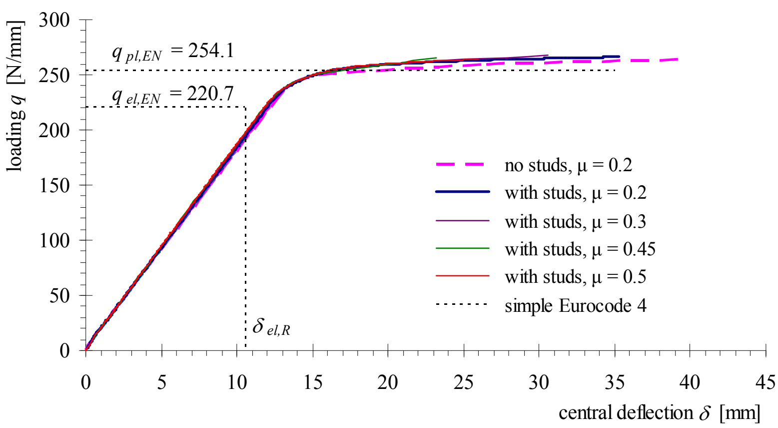

Fig. 12 Load-central deflection curves

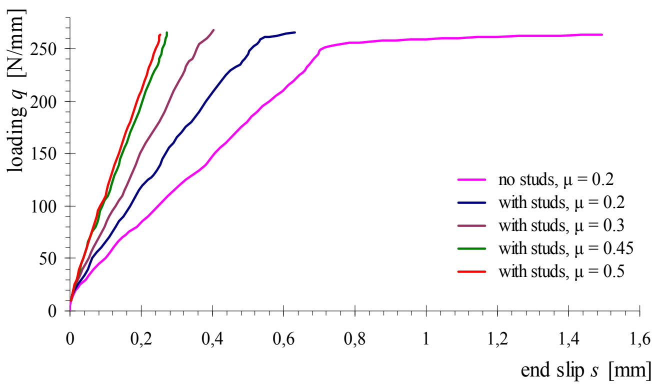

Fig. 13 Load-end slip curves

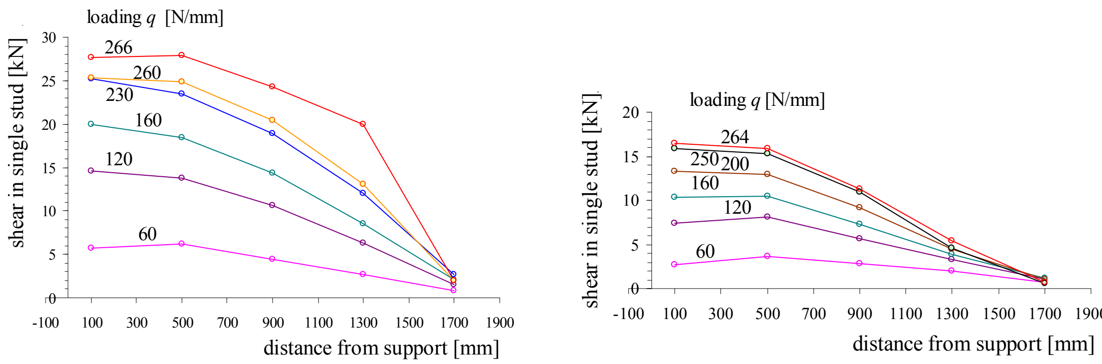

Fig. 14 Shear force in single studs (with the location of studs in curve breaks and spacing of 400 mm). Friction coefficient $\mu = 0.2$ (left) and $\mu = 0.5$ (right)

connection $\eta = n/n_f = 10/13 = 0.77$) are also given. It should be mentioned, that the ANSYS collapse moment ($M_{ANSYS} = 483.3$ kNm for $\mu = 0.3$) is greater than the Eurocode one ($M_{EN} = 457.3$ kNm) due to effect of confined concrete (see Elghazouli and Treadway 2008).

Fig. 12 demonstrates little effect of both the friction coefficients and headed studs in this case. On the other side the end slips (taken between bottom steel flange and concrete at the supports) significantly differ as shown in Fig. 13. Only in cases of $\mu = 0.2$ with/without additional studs the nonlinear slip collapse occurred, while for $\mu = 0.3 \div 0.5$ plasticity of steel or concrete failure led to collapse with still an elastic slip behavior.

Naturally, shear in stud connectors is decreased with greater friction coefficient, Fig. 14 (half span shown). With increasing loading the nonlinearity of the stud behavior is noticeable as seen particularly near supports.

Shear transmitted by friction on one side of the beam was received by integration of friction stresses in contacts. Comparison of the distribution along half span for beam without and with headed studs (located in distance x = 100, 500, 900, 1300, 1700 [mm]) is shown for friction coefficient $\mu = 0.2$ in Fig. 15. Clearly in location of studs the shear transmitted by friction also increased. At collapse the shear due to friction near the midspan rapidly increases following the beam plasticity.

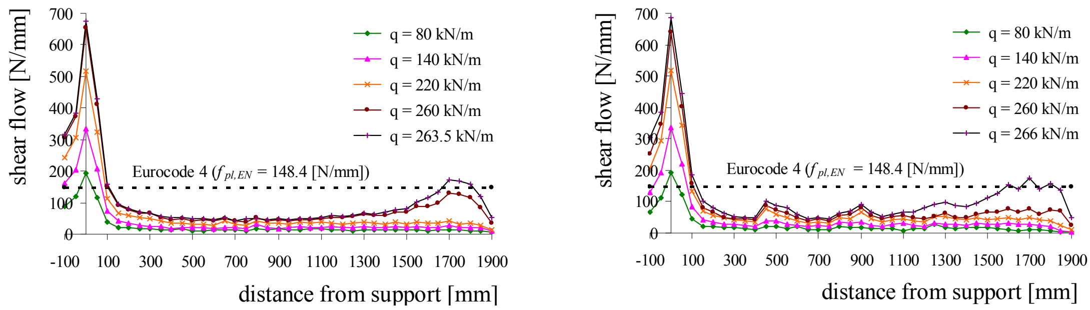

Fig. 15 Shear flow transmitted by friction in steel-concrete contact with friction $\mu = 0.2$.
Beam without studs (left) and with studs (right)

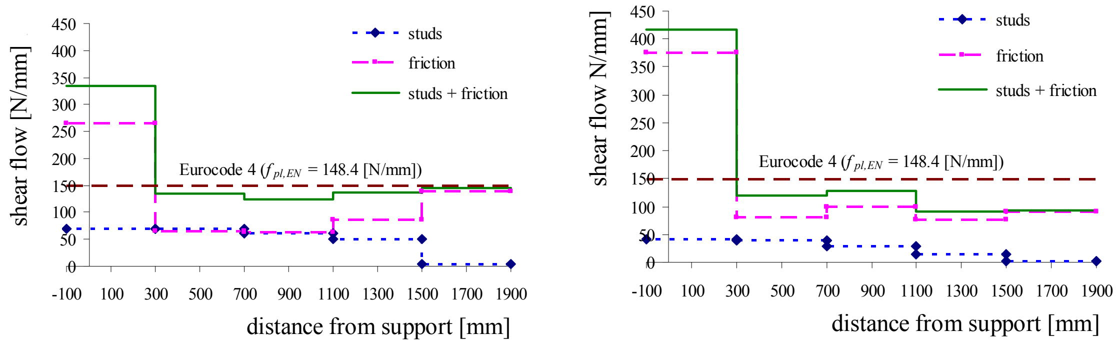

Fig. 16 Longitudinal shear transmitted by studs and due to friction.
Friction coefficient $\mu = 0.2$ (left) and $\mu = 0.5$ (right)

Finally the distribution of longitudinal shear between the part transmitted by friction and the part transmitted by studs at collapse loading was examined. To find this proportion the continuous distribution of the shear flow due to friction was rearranged into a stepwise one corresponding to shear volume in range of the respective stud (at spacing of 400 mm in position of 100, 500, 900, 1300, 1700 [mm]), see Fig. 16. Also shown is the simplified partial interaction calculation of the longitudinal shear according Eurocode 4 at collapse load, which gives uniform shear flow on one side of the beam $f_{pl,EN}$ = 148.4 N/mm. In the studied partially encased beam under bending the influence of friction on the transmission of the longitudinal shear between the steel and concrete is the dominant one and the more dominant the greater the coefficient of friction is.

4. Conclusions

Stud shear connectors of small diameters (10 and 13 mm) are frequently used in composite construction both for strength (the shear transfer and a prevention of possible web buckling) and assembly reasons (concreting procedures). Their use is effortless due to modest, light and advanced welding devices. However, because of rather small strengths the current standards do not assume their regular use in load-bearing structures and relevant design data are not provided.

Ordinary push-out tests with specified but common small diameter studs proved that the

Eurocode 4 and AISC formulas for strengths of common diameters (16÷25 mm) provide conservative values even for smaller diameters. Nevertheless, due evaluation of the tests allowed to determine higher characteristic values and full shear force-slip relationships for the specified 10 and 13 mm studs loaded in shear both for a design or numerical modeling.

Partially encased composite beams loaded in bending using such studs were studied numerically. Rather insignificant effect of studs welded to webs of such beams was confirmed by former tests. However, to determine role of a friction and local studs in these beams the numerical model was constructed in ANSYS software. 3D nonlinear analysis employed surface to surface contact elements and nonlinear springs to analyze the shear transfer between steel and concrete interface. The model was successfully validated using former tests. Nevertheless, the numerical analysis could be improved by using other than ANSYS software (Release 12, 2009), perhaps more suitable for detailed analysis of nonlinear behavior of concrete material. Parametric study with various friction coefficients resulted into the following conclusions for the studied beam:

- The friction between steel and encased concrete in common range (μ = 0.2÷0.5) is sufficient to ensure full shear interaction in the case of the simple composite beam under bending.
- Moment capacity is positively affected by an effect of concrete confined in the steel profile.
- End slip between steel profile and concrete evidently depends on the friction value, however, with small friction (when slipping is a danger) the shear studs ensure the required small end slip.
- Interaction of small diameter studs and friction in shear transfer indicates rather small influence of the studs, depending on value of the friction (approx. 40% with μ = 0.2 and 15% with μ = 0.5).
- At collapse the transfer of shear due to friction in the plasticized region increases.

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