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RESISTANCE BETWEEN CONCRETE SURFACES OF COMPOSITE MEMBERS

MARTIN KRIŽMA, JAROMÍR PETRŽALA AND MAREK KOVAČOVIC

The use of prefabricated structural elements and monolithic constructions made in different times has been ordinary for long decades mainly in the bridge construction. A bearing construction formed by a system of prefabricated pre-stressed beams and an additional monolithic slab constitutes a composite heterogeneous element with various features and with discontinuity at a contact area of the joined members. The mentioned issue relates to the new structures. In the case of already used objects, we meet this topic at damaged and consequently strengthened elements of structures. A loading history change of structural engineering objects and also an enhancement of traffic volume on bridge structures are frequent and relevant reasons of bearing structures strengthening. It concerns mainly linear concrete bearing elements. The main objective of such a formed construction is its monolithic behaviour and an enhancement of its resistance, while a shear resistance in coupling should exceed its flexural resistance and its transversal shear resistance too. The aim of the research program was an experimental and theoretic analysis of shear resistance in coupling on experimental damaged beams, which are needed to be repaired. The introductory part of the paper deals with the basis of coupling concrete/concrete and with factors determining its resistance. In the consequent experimental part, there are evaluated two--stage tests of beams. The first stage simulated the process of damage; the second stage simulated the process of strengthening. Within the both stages, the characteristics determining the both limit states were recorded. In the second stage, the accent was put on shear factors in coupling. The experimental results are compared with the results obtained by numerical simulations on the FEM basis. In the whole topic range, an attention was paid to an influence of a contact gap modification on shear resistance in coupling. After the theoretic-experimental analysis, we can state that the normative relations for the resistance calculations are markedly on the safety side.

Keywords: linear elements of concrete structures, coupling, strengthening, identification of failure processes on contact gap, numerical modelling

1. Introduction

The beginning of prefabrication since 1950s has had an important justification from the reason of fast and economically convenient construction. The prefabrication is closely related to the technology of coupling, which is an effective method of connection of mainly beam prefabricated elements with a monolithically manufactured slab. A coupling can be also applied in the case of slab elements, namely at utilisation of filigran prefabricated units or at additional strengthening of, e.g., floor structures by an over-concreted membrane. The progressive development of this technology in light of reinforced concrete objects construction eventuated to a significant improvement of prefabricated units, mainly in the bridge construction.

But the coupling has also disadvantages:
a) assurance of monolithic synergism (cross-section discontinuity),
b) enhanced requirements on transport of prefabricated units,
c) different ages of concretes (time dependent effects).

The issue connected with points a) and c) relates also to the strengthening of damaged structures, which are formed by linear reinforced concrete elements. We has been with a theoretic-experimental analysis of the linear elements strengthening and with interaction between a damaged and a strengthening element since 2008. The analysis has been performed within the frame of cooperation of the Institute of Construction and Architecture SAS in Bratislava and the Faculty of Civil Engineering University of Žilina. Some results were presented, e.g., in (Križma et al., 2012). In the former phase, we studied kinds of the strengthening itself. The strengthening of damaged elements was realized by an over-concreted slab or by the slab together with the reinforcing sheets from glass fibres (GFRP). Later, we have focused on the study of an influence of a contact – strengthened/strengthening element – type on resistance, concrete deformations, deflections, and way of failure and characteristics of the cracks development process. And it just forms a theme of this paper.

2. Theoretical study of composite behaviour

2.1Principle of coupling

Coupling of elements significantly increases a cross-section capacity just by avoidance of a contact surfaces mutual slip. There is no need of some modification of cross-section geometry, increasing of material quality or using of supplementary reinforcing elements. For illustration, we introduce a brief comparison of behaviour of non-coupled and coupled linear elements at load.

In the first case (see Fig. 1), the elements are freely laid on each other, so there is possible their mutual slip at the contact gap level. A flexural moment will be divided into two moments, in the ratio of flexural rigidities of the elements, which are acting on the separate parts of the structure. Each element is deformed apart, strains and stresses are proportional to a fibre distance from a mass centre of the given element. We can write

$$M = M_1 + M_2 \wedge \frac{M_1}{E_1 I_1} = \frac{M_2}{E_2 I_2} \Rightarrow \sigma_{i,max} = \frac{M E_i}{E_1 I_1 + E_2 I_2} \frac{h_i}{2} \quad (i = 1, 2) \tag{1}$$

where I_i is the moment of inertia of the i-th element's cross-section with respect to its own principal axis. Thus, an uncoupled composite element appears as an inconvenient solution for it does not utilize a whole possible bearing cross-section capacity.

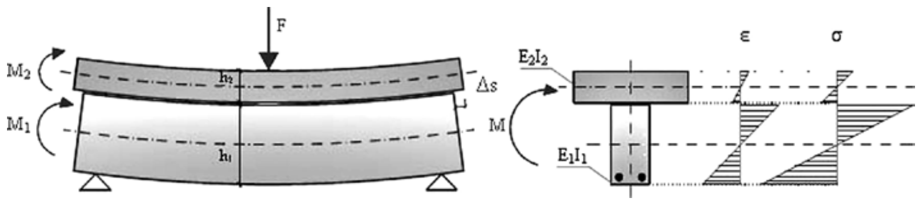

Figure 1. Behaviour of the uncoupled composite element – strain ε and stress σ distribution

In the case of coupled elements (see Fig. 2), a mutual slip of the elements at the level of their contact gap is avoided by any way. The interface takes over shear stresses whereby the internal forces transport between elements occurs. Such a composite element behaves like an element with a quasi-monolithic cross-section. At reinforced concrete structure, the coupling can be realized by coupling reinforcement, which passes across the contact gap, and/or by convenient modification of contact surfaces, which influences cohesion and friction. A coupling enables us by a relatively simple way to enhance a cross-sectional inertia moment of a whole structural element and thus also its flexural rigidity.

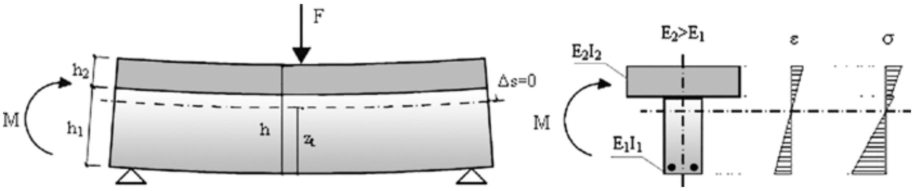

Figure 2. Behaviour of the coupled composite element – strain ε and stress σ distribution

In a coupled construction (see Fig. 2), a more effective distribution of stresses along the height of a cross-section occurs, thus the maximal stress value decreases. The maximal magnitudes of normal stresses in the marginal fibres can be calculated by the relations known from technical elasticity

$$\sigma_{1,max} = \frac{ME_1}{E_1I_1+E_2I_2} z_t \text{ , } \sigma_{2,max} = \frac{ME_2}{E_1I_1+E_2I_2}(h - z_t) \tag{2}$$

where I_i is now the moment of inertia of the i-th element cross-section with respect to the neutral axis of the whole composite cross-section and z_t denotes a position of this axis.

In Figure 3, there is depicted an illustrative numerical comparison of the normal stresses distribution at various coupling levels. Expediency of the coupled cross-section is evident from this comparison.

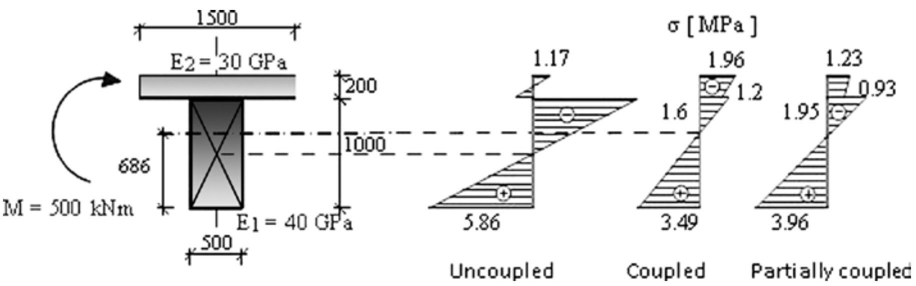

Figure 3. Comparison of stresses on uncoupled, coupled and partially coupled cross-section

The symbols presented in Figure 1 to 4 correspond to EN 1992 1-1 Eurocode 2 (2004).

2.2 Real level of knowledge of solved issue

In general, a longitudinal shear stress in an element arises owing to different flexural moments at cross-sections along an element. This stress is transferred by shear resistance of concrete at each level along the height of a cross-section, only at the contact gap it has to be transferred by factors participating in shear resistance at the coupling.

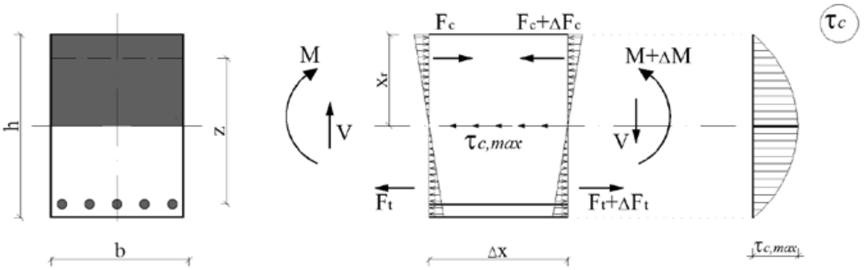

Figure 4. Loading effects – monolithic cross-section without a shear crack

Let us consider an element of a beam without a shear crack (low stresses domain) and with a monolithic cross-section as that one depicted in Figure 4 (Bilčík et al., 2008). Due to a flexural moment M, compressive and tensile normal forces act over the section. On the other end of the element, the opposed forces act, which are enhanced by the effect of an additional flexural moment $\Delta M = V\,\Delta x$. From the equilibrium condition of the element results that differences of these counter-acting forces have to be balanced by horizontal shear stresses at each level of the section. Thus, the horizontal shear stress in a distance z from the neutral surface can be calculated from the equation

$$\tau(z)b\,\Delta x = \int_z^{h/2} \frac{M+\Delta M}{I_y} z dA - \int_z^{h/2} \frac{M}{I_y} z dA \tag{3}$$

which yields

$$\tau(z) = \frac{1}{b\,I_y}\frac{\Delta M}{\Delta x}\int_z^{h/2} z\,dA = \frac{V\,S_y(z)}{b\,I_y} \tag{4}$$

where $S_y(Z)$ is a static moment of the corresponding section area in regard to the neutral axis. For the rectangular monolithic cross-section, Eq. (4) gives a quadratic distribution of the shear stress over the section with the maximum in the neutral surface (see Fig. 4).

At loading of bearing structures, significant transversal forces are generated mainly in the locations of supports. Due to these forces, shear cracks arise in an element. Their inclination corresponds to the principal stresses. Towards the centre of a beam, the cracks change into vertical flexural cracks induced by tensile normal stresses. So now consider a composite beam with the shear cracks as the one depicted in Figure 5 (Mac Gregor et al., 2000).

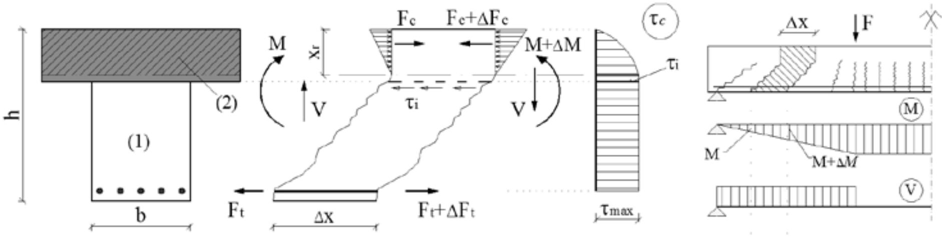

Figure 5. Loading effects – coupled cross-section with shear cracks

Now, the flexural moment M induces the compressive normal stresses only in the uncracked part (2) and the entire tensile force F_t is acting just in the tensile reinforcement at the bottom of the part (1).The horizontal shear stress has a constant value at each level of the cracked section until the interface and it can be simply calculated from the equilibrium conditions as

$$\tau b\, \Delta x = \Delta F_t = \frac{\Delta M}{a} = \frac{V\, \Delta x}{a} \;\Rightarrow\; \tau = \frac{V}{ba} \tag{5}$$

where a is an arm of the inner forces couple. In the case depicted in Figure 5, this shear stress constitutes the maximum value.

Shear resistance in the coupled elements interface is ensured by three simultaneously acting factors EN 1992 1-1 Eurocode 2 (2004), AASHTO, LRFD (1998).

1. Cohesion is an ability of two contact surfaces to resist a shear load without acting of any pressure force until the moment of a mutual slip independently of friction. It is caused by mechanical, chemical and dispersion bonds between surfaces. The coefficient c describing cohesion has to be determined experimentally. In accordance with a surface type, its value varies from 0.25 for the smooth surface up to 0.5 for the indented surface EN 1992 1-1 Eurocode 2 (2004).
2. Friction is dependent on an external normal force F_N across the interface. At a composite element, such a normal force is made by, e.g., a dead weight of a monolithic slab, a road layer, a cross wall, etc. A frictional force is given by the known relation $F_T = \mu F_N$, where μ is the coefficient of friction. This coefficient reaches values 0.5 – 0.9 in obvious concrete structures.
3. Shear coupling reinforcement forms the most significant factor that participates in shear resistance of a coupled element. According to the reinforcement ratio, it ensures (50 – 70) % of shear resistance in the coupled elements interface.

The essence of the coupling reinforcement function consists in its self-prestressing effect (Brugging, 1991). Due to roughness of the contact surfaces, opening of the contact gap happens at mutual slip of the elements. It induces a tensile force in the coupling reinforcement and a reaction on this force backward close to the contact gap (see Fig. 6). Thus the coupling reinforcement must be activated at first for it could perform its function. The reinforcement contributes to shear resistance by a frictional force induced by its compressive effect in the form of normal stress σ_c on the contact surface, which is obviously bounded by a width of a flange and by a mutual distance of the coupling reinforcement elements. If the reinforcement is fully utilized up to its yield strength, then the value of the frictional force owing to the reinforcement will be $V_S=\mu f_y A_s$, where f_y is the yield strength of the reinforcement and A_s is the area of the reinforcement crossing the interface.

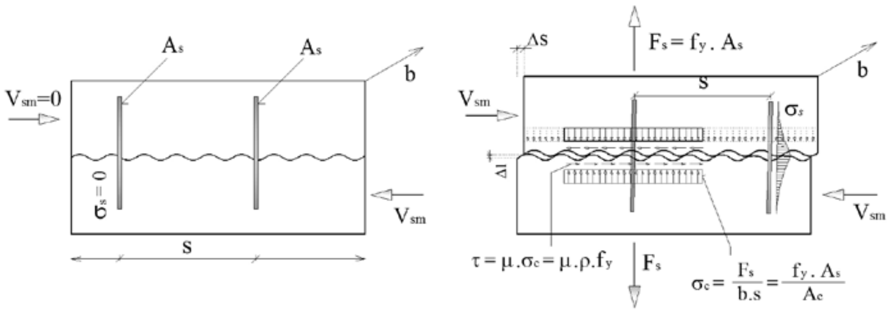

Figure 6. Principle of the coupling reinforcement effect

If the reinforcement is slant on the interface, the induced tensile force has two mutually perpendicular components (see Fig. 7). The first one, which is normal to the interface, has the same effect as in the case of the vertical reinforcement. The second component parallel to the interface acts directly against the shear load.

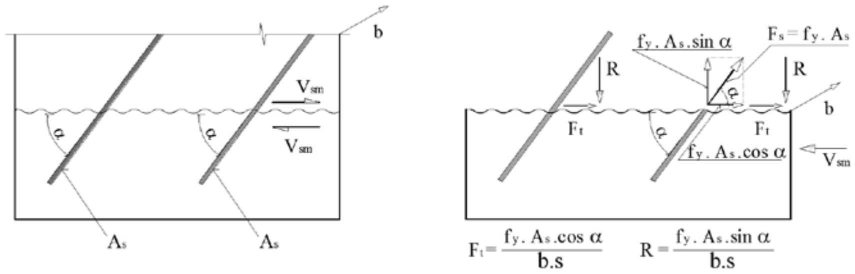

Figure 7. Principle of the slant coupling reinforcement effect

Thus, the total force and the corresponding stress, by which the reinforcement contributes to the shear resistance, is

$$V_S = f_y A_S(\mu \sin\alpha + \cos\alpha),\ \ \tau_S = f_y(A_S/A_C)(\mu \sin\alpha + \cos\alpha) \tag{6}$$

where A_c is the area of the contact. Consequently, the total design shear resistance at the interface can be calculated according to EN 1992-1-1 Eurocode 2 (2004) as

$$v_{Rd,i} = c f_{ctd} + \mu\sigma_n + f_y\rho(\mu \sin\alpha + \cos\alpha) \tag{7}$$

where c – cohesion coefficient (0.25 – 0.5)
f_{ctd} – tensile strength of concrete
μ – friction coefficient (0.5 – 0.9)
σ_{n} – normal stress on the interface
ρ – reinforcement ratio (A_S/A_C)
α – inclination of the reinforcement to the interface.

3. Experimental program

3.1 Description of experiments

For verification of the shear resistance in the contact of the coupled elements, there were made 6 beams with the total length 4 m and with the theoretical span l_t = 3.6 m – two beams with the coupling reinforcement as the reference ones – designation ST1 and ST2, two beams with the indented surface – Z1, Z2, and two beams with the rough surface – D1, D2. All these beams were being tested by the three-point flexural test in two stages. In the first stage, yet non-strengthened beams were being loaded up to the level γ = 0.7 (central loading force F = 350 kN). In the second stage, the beam had been strengthened by the coupling slab and then gradually loaded until the failure. The typical cross-section without the coupling reinforcement is depicted in Figure 8. Material characteristics of the beams and the coupling slabs are presented in Table 1.

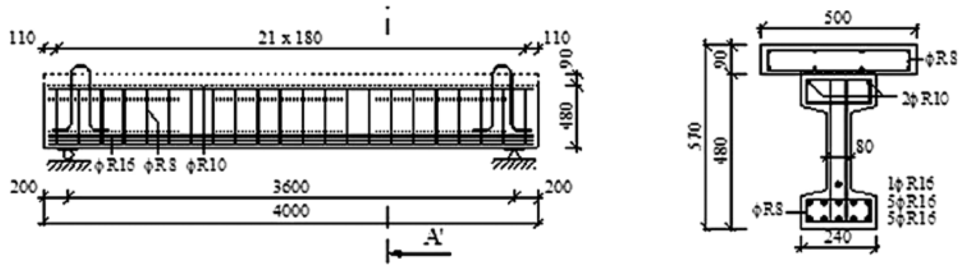

Figure 8. Geometry and scheme of reinforcing of the experimental beams

Table 1. Material characteristics of the beams and the strengthening slabs: cubic compressive strength of concrete $f_{c.c}$, flexural tensile strength of concrete $f_{ct.f}$ and Young's modulus of concrete E_c

Designation	Beam			Slab		
	$f_{c.c}$ [MPa]	$f_{ct.f}$[MPa]	E_c[GPa]	$f_{c.c}$ [MPa]	$f_{ct.f}$[MPa]	E_c[GPa]
ST1	59.87	5.98	39.26	61.22	6.00	41.13
ST2	62.47	6.72	39.73	61.44	6.83	40.00
Z1	62.42	5.67	39.73	58.90	6.59	40.00
Z2	63.23	5.90	39.73	57.80	5.86	40.00
D1	58.25	5.22	35.40	59.53	4.78	37.73
D2	58.12	5.43	36.03	59.14	4.82	37.70

At the tests, the following quantities were being recorded: loading force F, deflections along the beam, strains of the fictive girder system, mutual slip of the beam and the slab, characteristics of the cracks development process.

If the testing element was failed owing to exhaustion of its shear resistance in the coupling, the failure was progressive without time signalization. The shear resistance was calculated by the relation (7).

3.2 Experimental results

The shear failure in the contact gap appeared only in two cases – beams with the rough surface D1 and D2. At the indented beams Z1 and Z2, partial exhaustion of their shear resistance in the contact occurred, the failure simultaneously appeared also in the web in the middle of the beam shear span. The reference beams ST1 and ST2 failed by shear in the middle of the shear span, what also corresponded to assumptions at the coupling reinforcement design. The comparison of the experimental values at the failure with the numerical and design ones is presented in the next section. An influence of the contact type on characteristics of the serviceability limit state is analysed in a particular paper (Križma et al., 2013).

The typical ways of failure for the corresponding kinds of coupling are in Figure 9.

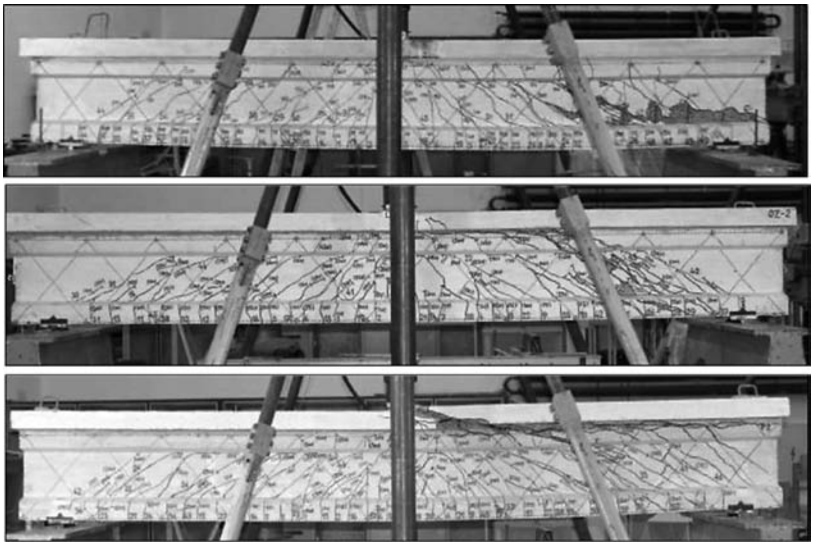

Figure 9. Shear failure. From the top: beam ST1 – failure of the web, beam Z2 – combination of failure web/contact, and beam D2 – failure in the contact gap

4. Comparison of experimental and theoretical results

4.1 Numerical analysis results

For complex verification of the coupled beams behaviour, the numerical analysis by the software ATENA 3D was performed (Červenka and Červenka, 2009; Kabele et al., 2005). The modelled beam was compounded from 3D elements tetrahedrons. Each element contained 10 nodes with three degrees of freedom. For a non-linear analysis, the material `3D Nonlinear Cementitous` was chosen as the model of concrete. Necessary material characteristics were obtained by laboratory measurements. The reinforcement was modelled by elements `Reinforcement` using a bilinear diagram with an increasing branch.

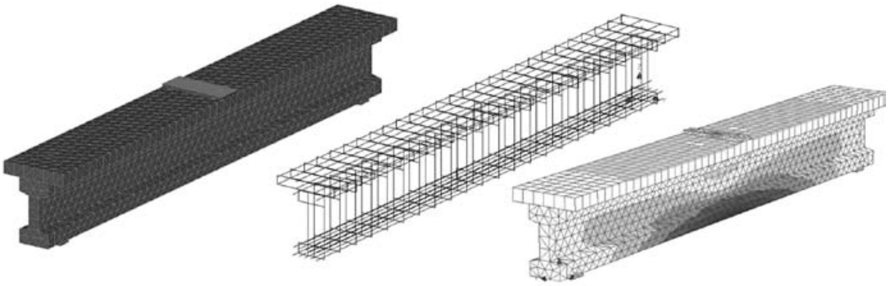

Figure 10. FEM model of the beam without the coupling reinforcement

The software ATENA uses for modelling of a contact between two materials the material element called "GAP", which simulates a real bond between surfaces. This element is defined by few parameters (cohesion, friction coefficient, tangential and normal stiffness of the contact) resulting from real measurements or from recommended values according to the surface modification of the contact gap.

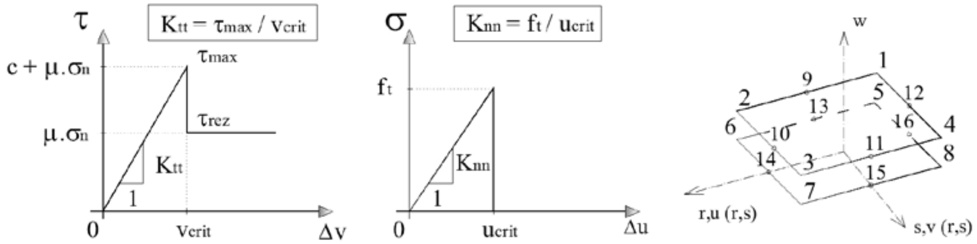

Figure 11. Tangential and normal stiffness of a contact and the contact element GAP

The tangential stiffness K_{tt} represents a value of a shear stress necessary for the unit length mutual slip. After releasing the contact due to overrun of its shear resistance ensured by cohesion and friction, this stiffness vanishes. If it is defined, the contact has the residual tangential stiffness $K_{tt,min}$. By analogy, the normal stiffness K_{nn} presents the relation of a tensile normal stress acting to the contact surface and a strain at which the mutual separation of the elements appears. When the normal stress exceeds the concrete tensile strength, the contact has the null normal stiffness. The contact element is represented by the pair of parallel surfaces with 8 or 16 nodes, according to the selected approximation.

Using the non-linear analysis, all considered surface modifications inclusive of the reference beams with the coupling reinforcement were modelled and the results were compared with the experimental analysis and with the design values (Kovačovic et al., 2013).

We can see from the diagrams in Figure 12 that the resistances of the beams without the coupling reinforcement are nearly equal. The resultant resistance is given by the resistance of the beam and the coupling slab, and these are very similar at all the beams. After exceeding the bond strength in the contact, there remains only a residual bond (it does not vanish entirely). Such a beam behaves like a partially coupled one and its resistance is limited by a capacity of the particular coupled elements.

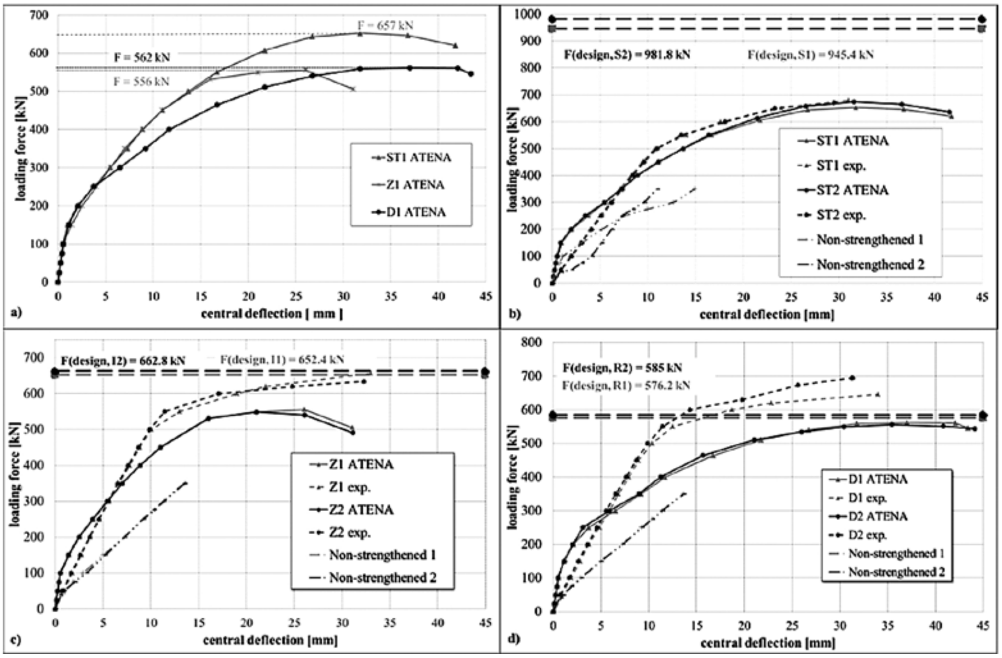

Figure 12. Work diagram – loading force vs. central deflection. Comparison of computational results for various coupling (a); comparison of computational and experimental results for ST (b), Z (c) and D beams (d)

Further, we can see that the resistances obtained by the computer analysis and by the experimental measurements are similar for all the contact surface modifications. However, the design shear resistances for D beams form approximately only 70 % of the really achieved values. The seeming fact, that the shear resistance of the elements without the coupling reinforcement is not significantly influenced by the contact surface modification, does not have to be true. This phenomenon occurred due to the weaker transversal shear resistance of the beam, when the shear resistance in the contact was not entirely exhausted, just partially, and the failure evolved to the diagonal shear crack. It is possible to assume that the concrete in the interface achieves the higher bond strength than the corresponding standard recommends. In the case of the rough beam D2 (Fig. 9c), the contact failed nearly along the whole element length, when closely the beam front (circa 500 mm from the support), the crack changed its direction into the diagonal crack. The shear resistance reached circa the value of 350 kN, which is nearly about 20 % higher than the corresponding design value.

4.2 Results on basis of standard recommendation

As we can observe from the values in Table 2, the beam has the smallest resistance in transversal shear due to an insufficient amount of stirrups in the beams. But the experimental results showed the other ways of failure. Design values of resistances were calculated according to EN 1992 1-1 Eurocode 2 (2004).

Table 2. Design, computational and experimental resistance of the coupled beams

Coupling type	EN 1992 1-1, Eurocode 2					
	Flexure	Coupling				
	$F_{fl.max}$ [kN]	c	μ	ρ_{sl} [%]	$V_{r.max}$ [kN]	$F_{sh.max}$ [kN]
ST1	706.4	0.45	0.7	0.18	472.7	945.4
ST2	707.8	0.45	0.7	0.18	490.9	981.8
Z1	702.2	0.5	0.9	0	326.2	652.4
Z2	708.1	0.5	0.9	0	331.4	662.8
D1	706.1	0.45	0.7	0	288.1	576.2
D2	706.2	0.45	0.7	0	292.5	585.0

Coupling type	ATENA 3D results			Experimental results		
	F_{num} [kN]	F_{num}/F_{exp} [%]	Type of failure	F_{exp} [kN]	F_{design}/F_{exp} [%]	Type of failure
ST1	652	95.9	Shear	680	103.7	Shear
ST2	674	100.6	Shear	670	105.6	Shear
Z1	556	84.6	Shear / Contact	657	99.3	Shear / Contact
Z2	549	82.8	Shear / Contact	635	104.4	Shear / Contact
D1	561	84.1	Contact	665	86.6	Shear / Contact
D2	556	80.0	Contact	695	84.1	Contact

It can be seen, that the resistance values according to the numerical analysis are in the best compliance with the experimental ones in the case of the reference beams with the coupling reinforcement ST. $V_{r,max}$ represents the transversal force at which the shear failure in the contact will appear and $F_{exp,max}$ represents the corresponding loading force. For the other types of coupling, the resistances were also in good compliance but with higher deviations (~ 5 – 20 %). It can be imputed to the fact that the cohesion parameters are estimated with a relatively bigger dispersion and they are significantly dependent on a production technology of the beam surface contrary to the reinforcement with relatively stable working characteristics. Even in the case of the beam D1 with the rough surface, the mutual slip did not appear at the contact surface level but in the domain of the \`healthy\` monolithic concrete. As if the contact surface cohesion was higher than the mutual cohesion of the monolithic concrete, that can be explained by exceedingly developed ability of binding of the particular elements of the concrete.

5. Conclusions and discussion

If the element is, from technological or another serious reasons, disconnected in the contact gap, except the mentioned loads also the examination of the shear resistance in the coupling is important. The aim of this study was to theoretically and experimentally analyse this resistance and to compare the obtained results. The shear resistance was verified on the coupled beams using the various factors influencing this resistance.

In the offered paper, the actual state of the theoretical analysis of the problem is documented. This analysis was verified by the trial prefabricated beams supplemented by the coupling slab with various surface modifications of the contact gap and also using the coupling reinforcement. Totally 6 beams have been tested. The two were reinforced by the coupling reinforcement; the others used only the cohesion and friction effects, which participate in the shear resistance in coupling. The numerical analysis was performed in the ATENA 3D software background.

The experimental measurements confirmed correctness of usage of the normative calculations of the shear resistance in the coupling. The results of the computer analysis and the theoretical analysis showed a good mutual accordance of the resistances and the failure ways. The commercial software on the FEM basis using a material and geometric non-linearity well described the real behaviour of the coupled beams and also the modelling of the contact gap was performed with the relatively good compliance with the real behaviour of the structure. The correctness of the coupled structures modelling is in a high degree influenced by the setting of the convenient contact parameters that affect the resultant resistance and deformation of the structure. In contrast to the numerical and experimental analysis, the normative calculations of the shear resistance in coupling offer more conservative results.

In the case of the composite beams without the coupling reinforcement, the sudden exhaustion of shear resistance in the contact occurs without any warning signal. Thus we can talk about a fragile failure. This phenomenon is caused just by the absence of the coupling reinforcement, by which it is possible to control the mutual slip of the coupled elements. Therefore, it is recommended to reinforce the contact gap at least to a minimal reinforcement ratio, even in the case, when the calculated resistance does not require its application.

Acknowledgements

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