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Behaviour evaluation of shear connection by means of shear-connection strips

Marian Rovnak† and Antonia Duricova†

Technical University of Kosice, Faculty of Civil Engineering, Vysokoskolska 4, SK-042 00 Kosice, Slovakia

Abstract. Comparison of behaviour of shear connections by means of shear-connection strips (perfobond and comb-shaped strips) and headed studs under static and repeated loading, possible failure modes of concrete dowels and ways of the quantitative differentiation of some failure modes are described in the paper. The article presents a review of knowledge resulting from the analysis of shear-connection effects based on tests of perfobond and comb-shaped strips carried out in the laboratories of the Faculty of Civil Engineering of the Technical University of Kosice (TU of Kosice) in Slovakia and their comparison with results obtained by other authors.

Key words: composite steel and concrete structures; perfobond strip; comb-shaped strip; failure modes; concrete dowel; push-out test; repeated load.

1. Introduction

Behaviour of composite steel-concrete structures is significantly affected by properties of shear connectors.

A revival of interest in composite steel-concrete bridges in the mid-1980s therefore naturally induces an increasing interest in efficient and effective shear connectors. In their research two tendencies are evident:

- more detailed examination of headed studs, e.g., headed studs of bigger diameters (Hanswille *et al.* 1988), bent, inclined (Pivonka *et al.* 2001), or horizontal studs (Breuninger 2001, Kuhlmann and Breuninger 1998), studs with elliptical shaft cross-section, double studs and studs made of high-strength steel (Hegger *et al.* 2001),
- design and analysis of alternative ways of shear connections, e.g., shear-connection strips of various shapes (Andrä 1985, Leonhardt *et al.* 1987, Kraus and Wurzer 1997, Duricova *et al.* 1998, Machacek and Studnicka 1999, Rovnak *et al.* 2000, Poot 2001), or thin-walled shear connectors (Fontana *et al.* 2001).

Headed studs were preceded by a variety of anchors and hoops from concrete reinforcement (Fig. 1a, b) and block connectors, most of which had to be combined with anchors and hoops to prevent the concrete slab from lifting due to shrinkage and thermal loading (Fig. 1c, d). Some of them can be seen even at present in a changed form as alternative shear connectors for headed studs.

†Associate Professor, Ph.D.

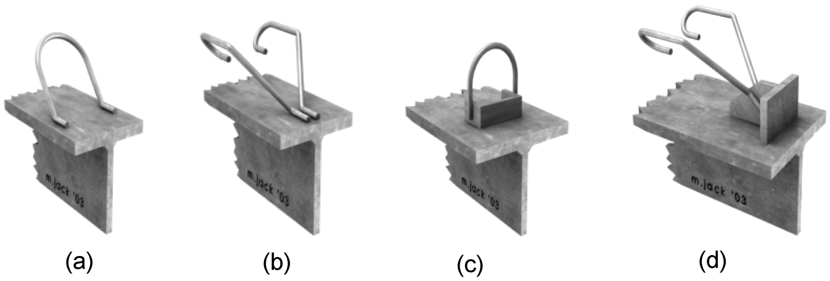


Fig. 1 Traditional shear connectors

Shear connection by means of headed studs, which is considered to be a standard technology with a high automation degree, leads to the following two unfavourable consequences:

1) As a shear connection by means of headed studs provides a discontinuous force transfer from concrete slab to flange of steel girders.
   Live loads cause that headed studs are subject to shear flow that can alternately change its direction. The weld of a headed stud is often the initial source of cracks spreading into the flange of main girder. In terms of fatigue failure, such a structural detail in composite steel-concrete girders is the most unsuitable one.
2) Resistance of a shear connection is determined by resistance of studs used in deck plates of common concrete grades and, therefore, an increase in bearing capacity of composite girders by the use of high-strength concrete is impossible to expect.

Many shear connectors, at present designed as alternative connectors to headed studs, resemble traditional shear connectors from the 1960s (STN 73 2089: 1961). To provide a connection preventing the slab lifting, concrete reinforcement in the form of stirrups or passed through openings in block connectors, or block connectors with inclined headed plates are used (Fig. 2).

Thin-walled shear connectors and saw-tooth connectors can be classified as alternative shear connectors. Nailed X-HVB shear connectors and trapezoidal strip connectors with perforated side walls were developed by the Hilti Corporation. Resistance of a shear connection by means of thin-walled shear connectors is, similarly to a shear connection by means of headed studs, determined by the strength of steel shear connectors (Fontana *et al.* 2001). It is, therefore, useless to use thin-walled shear connectors in high-strength concrete.

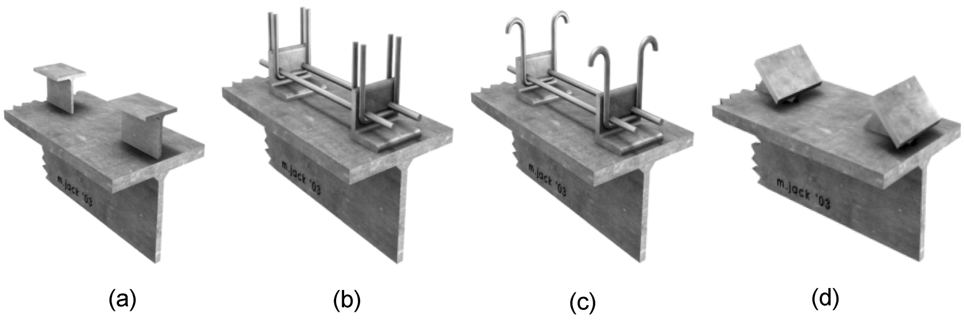


Fig. 2 New block shear connectors: (a) Galjaard *et al.* (1992), (b), (c) Ikeda *et al.* (2002), (d) Hegger *et al.* (2001)

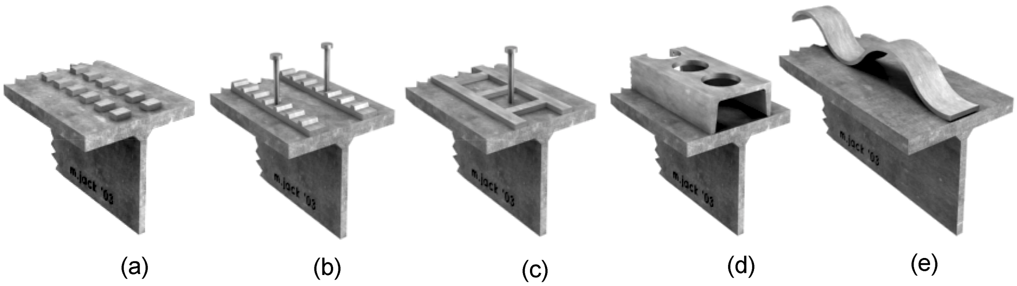


Fig. 3 Lying shear-connection strips: (a) Kato *et al.* (1985), (b) Leonhardt (1951), (c), (d) Andrä (1985), (e) Galjaard *et al*. (1992)

The new saw-tooth connectors were designed to transfer concentrated force effects. They are used in steel-concrete composite girders with truss walls to form joints between steel beams and concrete flanges and to anchor rope hangers, respectively (Schleich 2001, Schmid 2001).

Unlike headed studs, most of the traditional or new shear connectors (Fig. 1c, d, 2d) are not symmetrical with regard to the axis perpendicular to the shear flow direction. This results in different conditions of shear connections in case of the shear-flow-orientation change. Therefore, such shear connectors must be adjusted at least to the orientation of the prevailing shear flow.

This drawback does not occur in lying and vertical shear-connection strips. Lying shear-connection strips only provide shear connection in the longitudinal direction (Fig. 3), and that is why they always have to be combined with elements preventing the concrete slab from lifting.

One of the first lying strips combined with headed studs (Fig. 3b) was designed and used in practice by Leonhardt in 1938 (Leonhardt 1951).

If lying strips are formed, for example, as a perforated wall of a rolled U-beam (Fig. 3d), or they are without openings but corrugated (Fig. 3e), they enable connection in the vertical direction. In the first case, a thorough filling-in of the space between the rolled beam and the flange with concrete mixture can cause difficulties. In the latter case, the main disadvantage lies in short welded joints with notch effect.

To provide a shear connection effective in both longitudinal and vertical directions, the vertical perfobond strip (Fig. 4) developed in Germany (Andrä 1985) by the company of Leonhardt, Andrä und Partner in 1985, is a very simple solution. A detailed analysis of its behaviour was presented in Leonhardt *et al.* (1987).

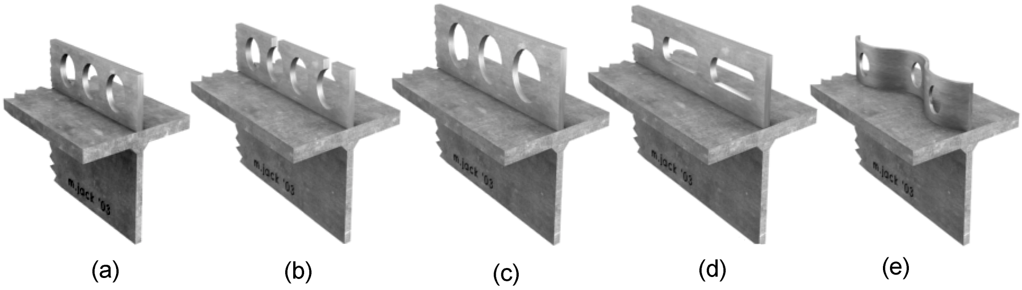


Fig. 4 Vertical shear-connection strips with openings: (a), (b) Leonhardt *et al.* (1987), (c) Kraus and Wurzer (1997), Ushijima *et al.* (2001), (d) Poot (2001), (e) Galjaard *et al.* (1992)

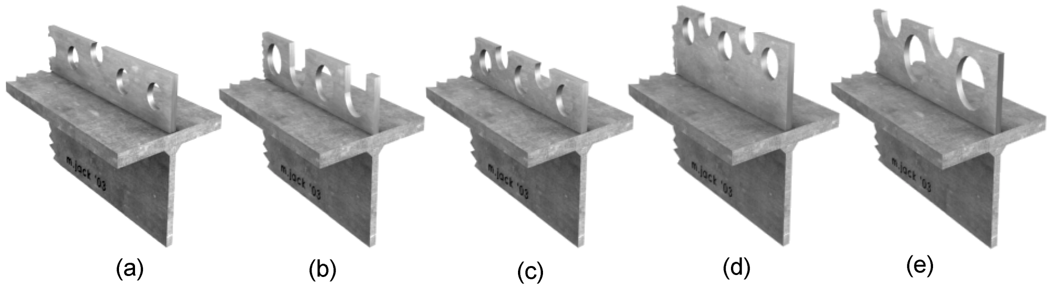


Fig. 5 Vertical shear-connection strips with openings and notches: (a) Andrä (1990), Roberts and Heywood (1995), (b) Oguejiofor and Hosain (1994), (c), (d), (e) Studnicka and Machacek (2002)

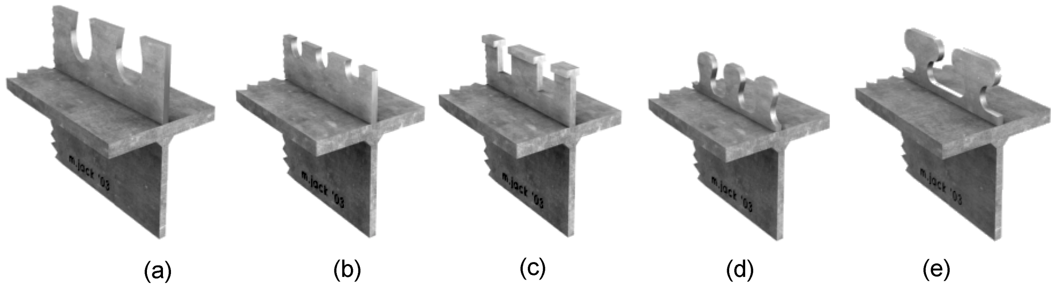


Fig. 6 Vertical shear-connection strips with notches and comb-shaped strips: (a) Kraus and Wurzer (1997), (b), (c) Hegger *et al.* (2001), (d), (e) Rovnak *et al.* (2000)

Investigation of shear-connection strips has gradually continued also in other countries. In Canada the first preliminary results were obtained in Antunes (1988), and their analysis together with other results was published in Veldana and Hosain (1992). In Australia results were published in Roberts and Heywood (1992), in Japan in 1994 (Ushijima *et al.* 2001), and in the Czech Republic in Studnicka, Machacek and Peleska (1996), research started in 1994. In the Slovak Republic the first results of shear-connection tests were published in Nad, Duricova and Rovnak (1997), research between the company of Engineering Structures, Inc. of Kosice and the Technical University of Kosice started in 1995.

Designed and experimentally as well as theoretically evaluated were shear-connection strips of various shapes:

- strips (straight or corrugated) with openings only (Fig. 4),
- strips with openings and notches (Fig. 5),
- strips with notches only (Fig. 6a-c),
- comb-shaped strips (Fig. 6d-e),
- combined strips allowing double shear connection (Fig. 7),
- shear-connection strip as a perforated web edge of a girder without the upper flange (Fig. 8a),
- shear-connection strip as a perforated and corrugated web edge of a girder without the upper (or lower) flange (Fig. 8b),
- perforated edge of a corrugated web with two perfobond strips (Fig. 8c).

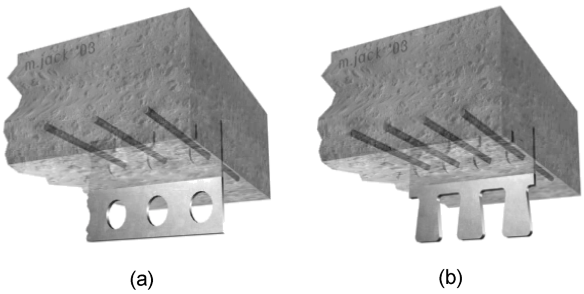


Fig. 7 Doble-shear-connection strips (Poot 2001)

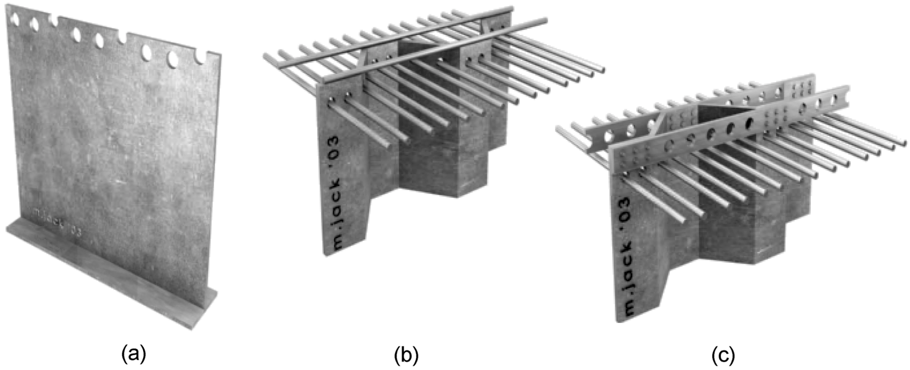


Fig. 8 Shear-connection strips as perforated edge of a straith or corrugated girder web: (a) Roberts and Heywood (1995), (b) Sakurada *et al.* (2001), (c) Sugimoto *et al.* (2003)

2. Shear connection by means of perfobond strips, comb-shaped strips and headed studs

2.1. Shear connection under static load

Load-slip diagrams of the tested shear connectors (Fig. 9) show that there is an apparent “boundary” (dashed line in Fig. 9) between the behaviour of samples with perfobond strips or headed studs on the one hand, and samples with comb-shaped strips on the other hand.

The two shear connections by means of perfobond strips and headed studs can be considered as equivalent and, obviously, more rigid than the shear connection by means of comb-shaped strips.

Failure of a shear connection by means of perfobond strips or headed studs arises when the slip is 3-5 mm, while by comb-shaped strips it occurs when the slip exceeds 5 mm.

Based on the regression analysis (Fig. 10) of the obtained results, it is possible to consider the behaviour of shear connections by means of “A” and “B” comb-shaped strips as equivalent up to the ratio $P/P_{u,\,exp}$=0.5, while at higher values the shear connection by means of the “B” strip appears to be less rigid.

At the slip value corresponding the failure of shear connections by means of perfobond strips or headed studs, the comb-shaped strips reach only about 85% of their resistance.

Experimental resistances of perfobond-strip shear connections were expressed as the shear force per

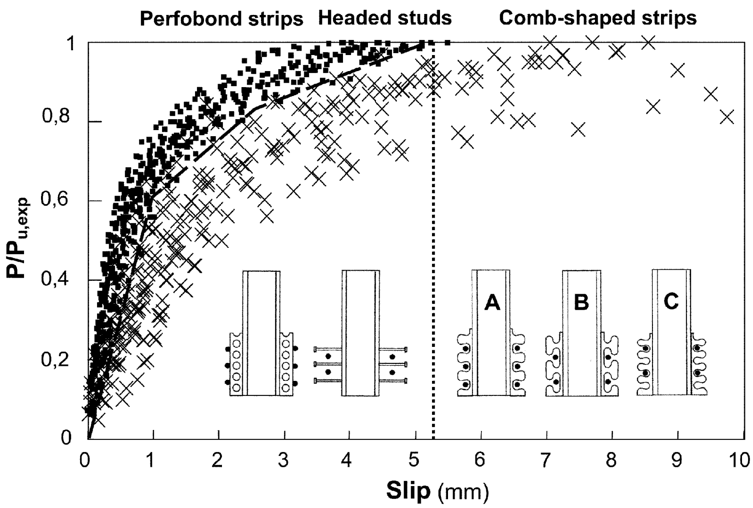


Fig. 9 Load-slip diagrams for shear connections by means of headed studs, perfobond and comb-shaped strips

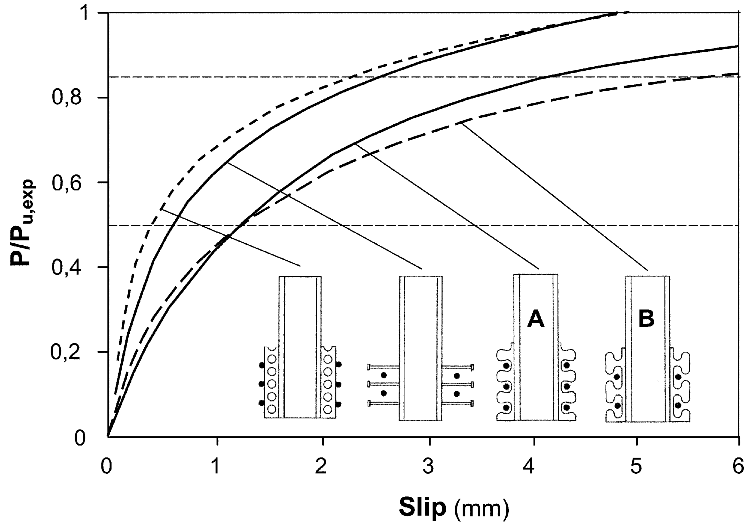


Fig. 10 Load-slip regression curves of tested types of shear connectors

concrete dowel (kN/concrete dowel) or shear flow (kN/m) in relation with factors $d^{2}f_{cm}$ (Figs. 11, 13), $d^{2}(t/d)^{0.5}f_{cm}$ (Figs. 12, 14) and f_{cm} (Fig. 15). Here d (mm) is the concrete dowel diameter, t (mm) is the strip thickness and f_{cm} (N/mm^2) is the mean value of cylindrical concrete strength.

Although results obtained at the TU of Kosice create just a small part of the diagram, it is obvious from Fig. 11 that an adequate correspondence with results by Andrä (1985), Leonhardt *et al.* (1987), Roberts and Heywood (1995), and Ushijima *et al.* (2001) exists.

Experimental resistances (Studnicka and Machacek 2002) of shear-connection strips without reinforcement are substantially lower and less affected by the concrete strength than other resistances, and resistances of strips with transverse reinforcement in openings are also lower.

Results according to Poot (2001) adequately correspond with results of other authors only when

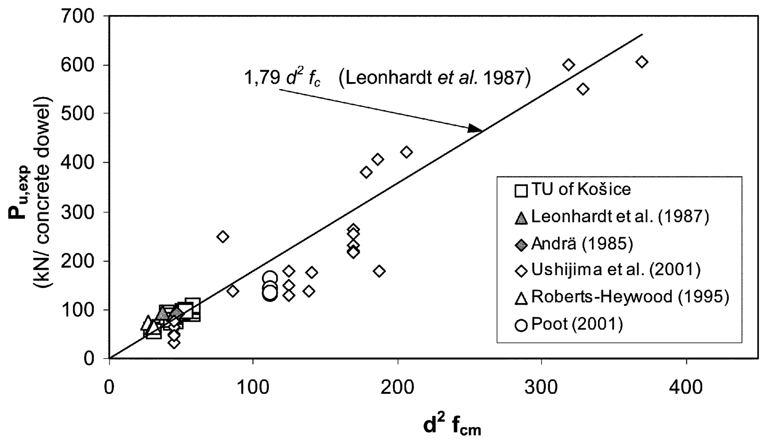


Fig. 11 Experimental concrete dowel resistance $P_{u,exp}$ versus factor $d^2 f_{cm}$

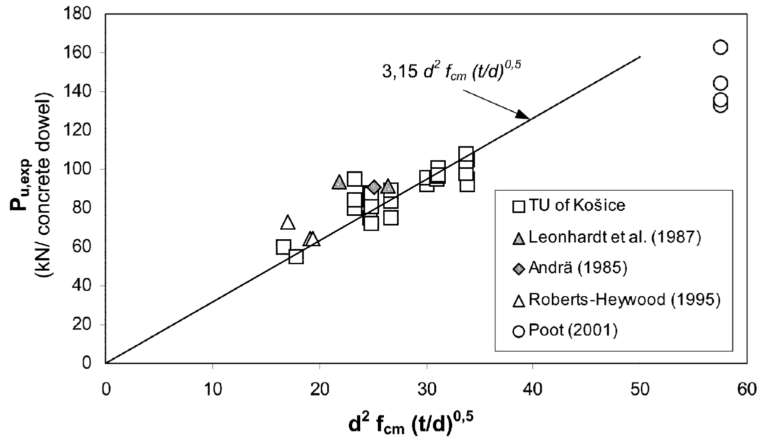


Fig. 12 Experimental concrete dowel resistance $P_{u,exp}$ versus factor $d^2 f_{cm}\,(t/d)^{0,5}$

demonstrated as the shear force per concrete dowel in relation with the factor $d^2 f_{cm}$ (Fig. 11). In other cases the resistances quite significantly differ.

The overall results demonstrated in relation to a traditional factor, the concrete strength, seem to be the most consistent (Fig. 15).

Dropped connectors (Fig. 6a) tested by Kraus and Wurzer (1997) resemble the comb-shaped strips (Figs. 6d, e) designed at the TU of Kosice and they are also very similar to double shear connectors (Fig. 7b) dealt with by Poot (2001).

Samples according to Kraus and Wurzer (1997) consisted of concrete blocks of 180 or 220 mm thickness. Transverse reinforcing bars were passed through strip notches and their total cross-section area in each block was 300 and 500 mm^2, respectively.

The double shear connection by Poot (2001) was created in such a way that on each of both sides of the steel element a pair of perfobond strips with four oval openings was welded. Every concrete block was equipped with a built-in shear strip in the centre of its width centre (Fig. 7) and, thus, the strip was placed between the strips on the steel element. Space between the steel element flange and the prefabricated concrete block was filled with grout.

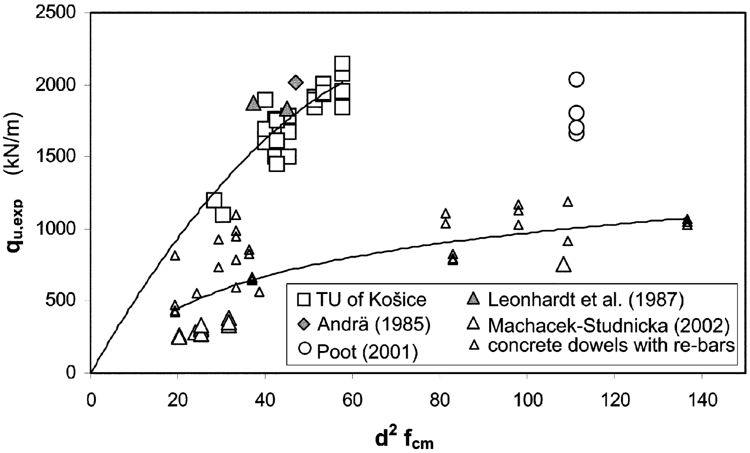


Fig. 13 Experimental concrete dowel resistance $q_{u,exp}$ versus factor $d^2 f_{cm}$

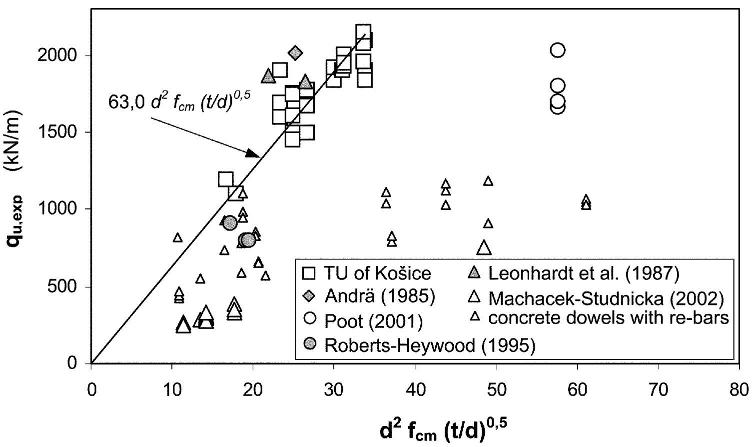


Fig. 14 Experimental concrete dowel resistance $q_{u,exp}$ versus factor $d^2 f_{cm}$ $(t/d)^{0,5}$

Fig. 16 illustrates a comparison of experimental resistances of comb-shaped strips “A” and “B” and resistances according to Kraus and Wurzer (1997) and Poot (2001) in relation to the mean cylindrical concrete strength (cylindrical strength was obtained by the conversion f_{cm}=0.8 $f_{c,cube,m}$). Experimental resistances of comb-shaped strips adequately correspond with experimental resistances according to Kraus and Wurzer (1997) for lower cylindrical strength values.

It can be seen that the shear-connection resistance according to Poot (2001) is the lowest. This supports the statement (Poot 2001) that the resistance of a double shear connection is approximately by 25% lower than of a shear connection in case of concrete blocks produced in-situ.

As quite interesting can be regarded the comparison of shear-connection-strip test results by means of the relations between relative shear stress $P_{u,exp}/A_S/f_{cm}$ and the A_S/A_C ratio (Fig. 17). Here $P_{u,exp}$ is the experimental concrete dowel resistance, A_S is the size of two shear areas of the concrete dowel, A_C is the contact area expressed as the size of the vertical projection of the contact surface of the concrete dowel and the shear-connection strip (Fig. 18).

It was shown that if A_S/A_C increase, the relative shear stress of concrete dowels is decreasing, gradually ceases to change and, finally, attains its constant, minimum value.

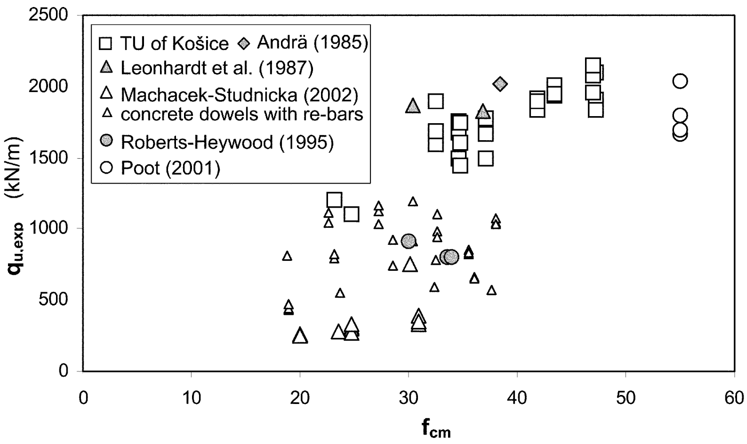


Fig. 15 Experimental concrete dowel resistance $q_{u,exp}$ versus factor f_{cm}

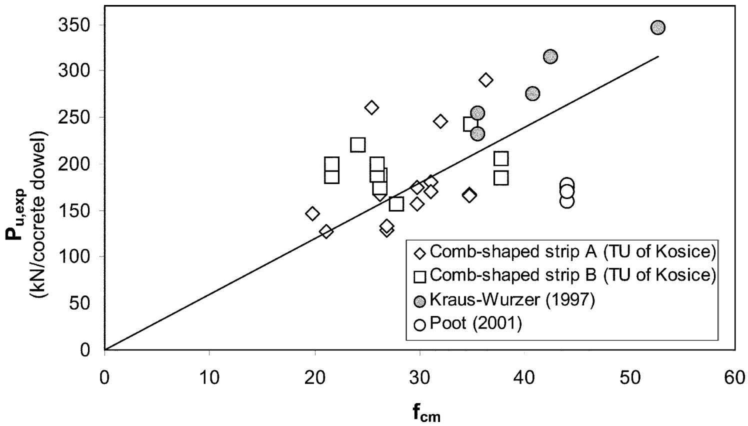


Fig. 16 Experimental concrete dowel resistance $P_{u,exp}$ versus factor f_{cm}

2.2. Shear connection under repeated loading

Till present times, only few tests dealing with shear-connection strips under repeated loading were carried out, e.g., Leonhardt *et al.* (1987), Klaiber and Wipf (2000), Studnicka (2003).

The results of perfobond-strip tests were first published in Leonhardt *et al.* (1987). Here shear connections by means of perfobond strips were compared with the shear connection by means of headed studs ∅ 22 mm according to Roik and Hanswille (1987), because the local contact stress at the stud footing was approximately equivalent with the contact stress of the concrete dowel.

Tests of perfobond and comb-shaped strips under repeated loading were carried out at the TU of Kosice in Rovnak and Duricova (1998). Chosen test parameters were similar to those in Leonhardt *et al.* (1987); frequency of repeated loading was 3 Hz, total force acting on each sample was 530 kN, the maximum and minimum force acting on a concrete dowel was 26,5 kN and 2.5 kN, respectively, i.e., the force amplitude onto a dowel was approximately 24 kN. The shape of the tested shear-connection strips is showed in Fig. 19. Since none of the samples failed under repeated loading during the tests, they were subsequently submitted to standard static push-out tests.

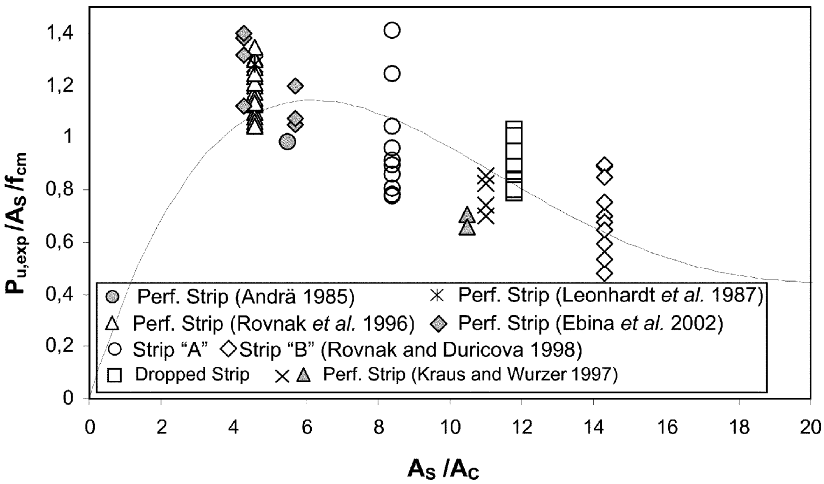


Fig. 17 Shear stresses versus A_S / A_C ratio

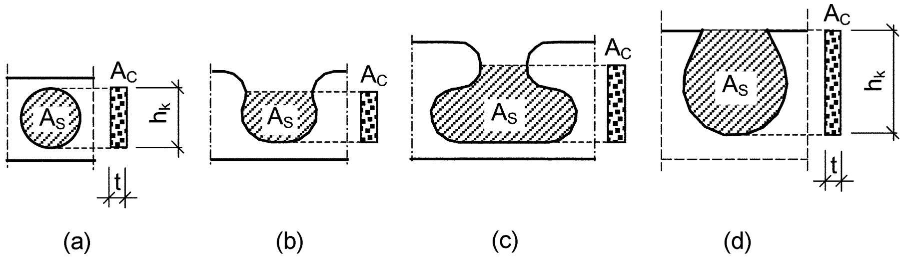


Fig. 18 Shear area A_S and contact area A_C for: (a) perfobond strip (b) comb-shaped strip "A", (b) comb-shaped strip "B", (d) dropped strip (Kraus and Wurzer1997)

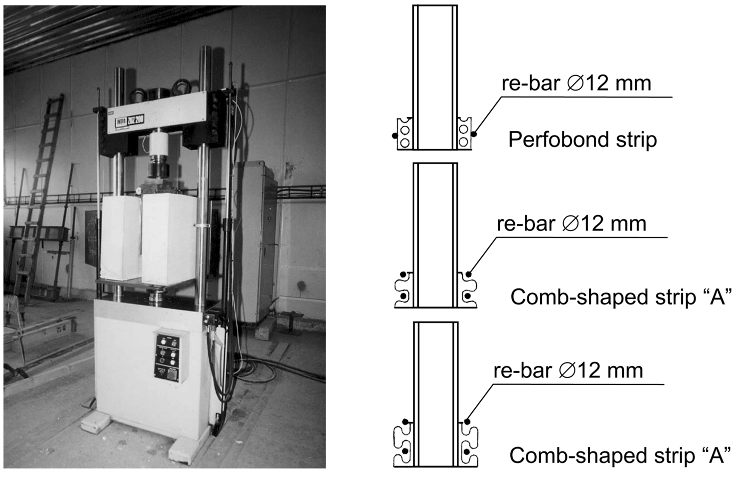


Fig. 19 Comb-shaped strips tested under repeated loading

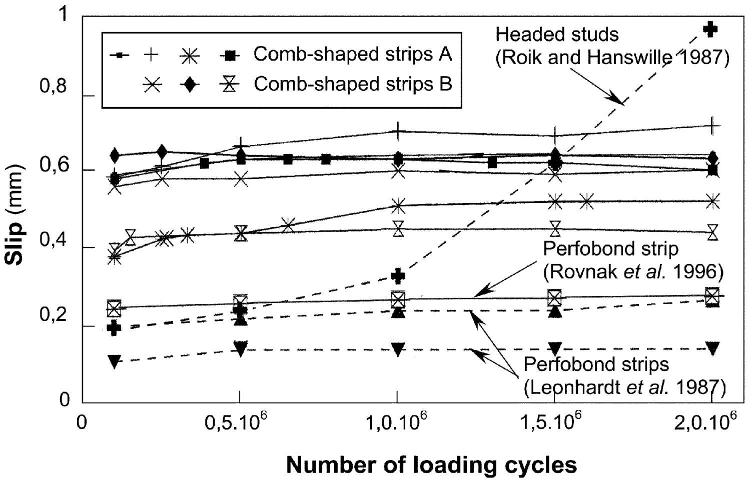


Fig. 20 Slip versus number of loading cycles for perfobond and comb-shaped strips

Fig. 20 illustrates a comparison of the test results according to Leonhardt *et al.* (1987) and Roik and Hanswille (1987) with the results obtained at the TU of Kosice.

As it is obvious from Roik and Hanswille (1987), at the shear connection by means of headed studs the slip rapidly increases after approximately 10^6 load cycles and the failure arises after approximately 2.5×10^6 cycles.

Slip caused by repeated loading is almost independent of the number of loading cycles. This is valid for both the perfobond and comb-shaped strips. The difference in their behaviour rests in the size of the initial slip, which is greater for comb-shaped strips.

The tests under repeated loading proved that the shear connections by means of perfobond or comb-shaped strips are not as sensitive to fatigue failure as the shear connection by means of headed studs.

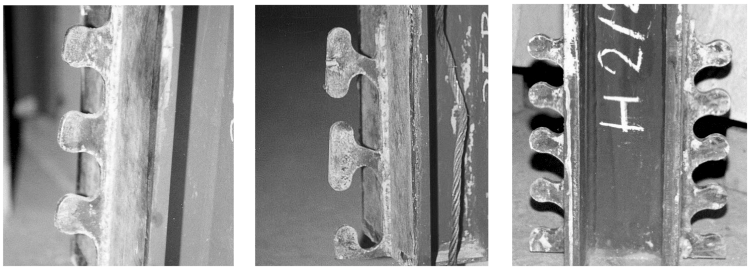
3. Failure modes of concrete dowels

Crucial factors affecting the failure mode of a shear-connection strip include thickness, size (and shape) of concrete dowels, amount of reinforcement in the direction perpendicular to the shear-connection strip and the concrete strength. The failure modes do not appear in isolation, one of them is usually the initial one and in the process of failure some other occur.

Tests of shear connections by means of vertical shear-connection strips (predominantly perfobond strips) showed that the collapse of the shear connection can be caused by:

- shear failure of concrete dowels,
- shear failure of perfobond strips between openings,
- crush of concrete dowels,
- split of concrete dowels,
- bending of teeth of comb-shaped strips,
- split of concrete blocks, and
- shear failure of concrete blocks.

Shear failure of a concrete dowel on two shear areas, which was historically the primal assumption



(a) (b) (c)

Fig. 21 Deformation of comb-shaped strips: (a) strip “A”, (b) strip “B”, (c) strip “C”

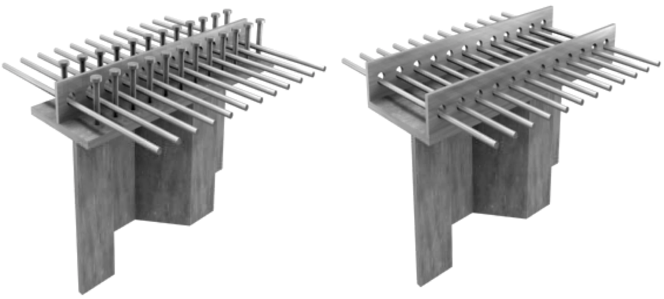
for determination of the concrete-dowel resistance (Leonhardt *et al.* 1987), can be considered to be a typical failure mode of the perfobond-strip concrete dowel.

A typical phenomenon at the collapse of a comb-shaped-strip shear connection is bending of its steel teeth (Fig. 21), which is accompanied by local crushing of concrete dowels. Thin shear-connection strips with openings close to each other fail due to shear between the openings (Leonhardt *et al.* 1987). In case of the greater distances between openings, the collapse is initiated due to the split of concrete dowels (Ushijima *et al.* 2001).

In relatively thick strips dowels fail predominantly due to shear, even though dowel failure due to crush on the contact area with the steel strip is also likely to occur.

From the first tests of shear connections by means of perfobond strips carried out by Leonhardt *et al.* (1987) it is evident, that they aimed at verification of the shear connections, the basic element of which was a concrete dowel. The role of reinforcement in concrete blocks was “just” to eliminate an inevitable and unfavourable phenomenon - splitting stresses over the shear-connection strip built in the concrete block. As stated in Leonhardt *et al.* (1987), reinforcement is usually placed on the perfobond strip, not through its openings.

A reinforcing bar placed in a dowel of a thin shear-connection strip eliminates splitting stresses. When placed in a dowel of a relatively thicker strip, it increases its shear resistance. In both cases crush of concrete between the reinforcing bar and the opening edge, and bar deformation occur at a large slip (Fig. 3b).



(a) (b)

Fig. 22 Perfobond strips with re-bars in every opening: (a) Kosa *et al.* (2002), (b) Ebina *et al.* (2002)

Transverse reinforcement in concrete dowels contributes, of course, to an increase in the resistance and mainly in deformability of the shear connection.

An arrangement where openings in the perfobond strip are relatively small and the reinforcing bar is passed through each opening (Fig. 22) can be regarded rather as a shear connection by means of concrete reinforcement than by concrete dowels. In such a case the concrete dowel with its original function does not almost exist.

A larger amount of reinforcement passed through strip openings can be the cause of shear failure at the large part of the concrete block adjacent to the perfobond strip (Studnicka and Machacek 2002). It is the consequence of the dominant shear-connection effect of reinforcing bars.

In some push-out tests, (e.g., Oguejiofor and Hosain 1992, Veldana and Hosain 1992, Studnicka and Machacek 2002), concrete blocks were split in the plane of the shear-connection strip. This can occur when blocks are thin and insufficiently reinforced, which, however, should not and cannot be regarded as a typical failure of shear connections by means of perfobond strips.

Not only concrete blocks but also slabs of real structures must be equipped with a sufficient amount of transverse reinforcement (Leonhardt *et al.* 1987) to avoid their splitting in any possible cases.

This has to be emphasized, as the concrete blocks in tests Oguejiofor and Hosain (1992) and Veldana and Hosain (1992) in spite of being reinforced in accordance with the standard reinforcement of slabs in buildings, failed due to splitting.

In the push-out tests according to Andrä (1985), Leonhardt *et al.* (1987), Roberts and Heywood (1995), as well as in tests carried out at the TU of Kosice, in which transverse reinforcement did not pass through strip openings but just touched the strip, failure due to splitting of blocks did not occur at all.

4. Quantification of failure modes

Quantitative differentiation of basic failure modes of concrete dowels (without reinforcing bars) by shear, crush or split is important in designing of shear connections from several aspects, for example from the point of view of anticipation of the initial failure mode of concrete dowels that could lead to the collapse of the shear connection.

To predict the kind of the failure mode of concrete dowels that is likely to occur, the concept of “small” and “big” concrete dowels was suggested in Kraus and Wurzer (1997). While small dowels usually fail due to shear, the big ones usually due to splitting combined with local crushing. The boundary between small and big dowels was specified either as 43 mm in diameter or the width of the concrete dowel.

A combined split-shear failure of a non-reinforced concrete dowel was taken into account in Ushijima *et al.* (2001) by the strip thickness and the dowel diameter t/d ratio in the formula for the estimation of the dowel resistance. However, the boundary between the particular failure modes was not specified.

To differentiate dowel failures due to shear or crush, the same t/d ratio was used in Furtak (1999). Here, from the formulas expressing the shear resistance or the crush resistance (including contributions of cohesion between concrete and either the flange or the perfobond strip) it follows that the crush resistance of a perfobond strip is decisive (i.e., is lower) when $t/d < 1.32 / (f_c)^{0.5}$ in comparison with its shear resistance.

From the point of view of concrete-dowel failure a more accurate differentiation was published by Rovnak, Duricova and Ivanco (2000). It was based on relations between the relative shear stress

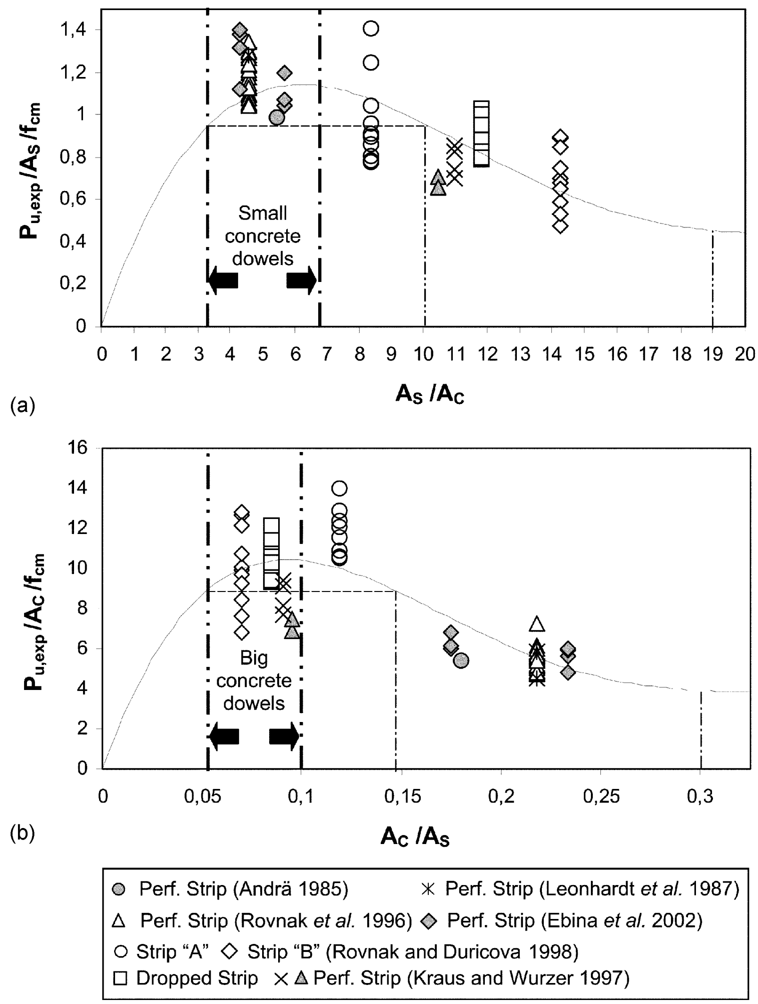


Fig. 23 Differentiation of small and big concrete dowels: (a) relative shear stresses versus A_S/A_C ratio, (b) relative contact pressure stresses versus A_C/A_S ratio

$P_{u,exp}/A_S/f_{cm}$ and the A_S/A_C ratio (Fig. 23a), and between the relative contact pressure stress$P_{u,exp}/A_C/f_{cm}$ and the A_C/A_S ratio (Fig. 23b).

The left boundary of the small-concrete-dowels zone in Fig. 23(a) is drawn for the ratio A_S/A_C = 3.333, because the contact pressure stress practically does not change for the ratio $A_C/A_S \geq 0.3$ in Fig. 23(b).

Similarly, as the shear stress does not change for the ratio A_S/A_C = 19.0 in Fig. 23(a), the left boundary of the big-concrete-dowels zone in Fig. 23(b) is illustrated for the ratio A_C/A_S = 0.053.

The right boundary of the small-concrete-dowels zone is determined for the ratio A_S/A_C = 6.8 in Fig. 23(a), which corresponds with the ratio A_C/A_S = 0.147 for the value of contact pressure stress which is at the left boundary of the big-concrete-dowels zone in Fig. 23(b).

In like manner can be determined the right boundary of the big-concrete-dowels zone is for the ratio A_C/A_S = 0.099 on the basis of the ratio A_S/A_C = 10.1 in Fig. 23(a) for the value of the shear stress which is at the left boundary of the small-concrete-dowels zone.

5. Conclusions

Shear connections by means of perfobond strips rightfully deserve attention because in comparison with the traditional shear connection by means of headed studs they seem to be more suitable from several points of view:

1) point of view of resistance:
   - resistance of concrete dowels is a crucial factor in shear connection failure, which makes it possible to use high-strength concrete,
2) point of view of the fatigue strength and durability:
   - shear connection provides continuous shear transfer to the flange of the steel girder, welded joint of a shear-connection strip with the flange is substantially a more suitable structural detail than that of a headed-stud joint,
   - slip between the concrete block (deck plate) and the steel girder does not depend on the number of repeated load cycles (on the contrary, slip rapidly increases when headed studs are used),
3) point of view of production and the assembly method of structures:
   - shear strip is the product of a construction company and in comparison with headed studs (in addition to other advantages also owing to their small height) enable high variability in designing and simplify the assembling of precast composite structures,
   - perfobond strips can function as stiffeners, and, therefore, it is not necessary to add further stiffeners onto the sheet of a steel-concrete deck plate.

The properties above mentioned predetermine this type of shear connectors for being used mainly in composite steel-concrete bridges, which are (in the case of medium and large spans) often more economical than pre-stressed concrete bridges.

It is possible to claim that the term of small concrete dowels can be used when $3.33 < A_S/A_C < 6.8$, big dowels require $10.1 < A_S/A_C < 19$. While in the first case shear failure occurs, in the second one splitting and crushing arise. Both failure modes of concrete dowels overlap in the interval $6.8 < A_S/A_C < 10.1$.

Shear failure of concrete dowels is considered to be a suitable mode. It, however, only concerns small dowels, which are typical mainly for perfobond strips. Comb-shaped strips, on the other hand, are definitely more advantageous than perfobond strips in terms of production expenses. In comb-shaped strips, even if of relatively small height, bigger concrete dowels are likely to be formed. These, however, tend to fail due to splitting combined with local crushing.

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