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Perforated shear connectors

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Abstract. Perforated shear connectors currently used in composite steel and concrete structures are described and evaluated. Modifications of the perforated connector suitable for common use in civil and bridge engineering are proposed. The connectors were tested in laboratories of CTU Prague for shear load capacity. Push tests of connectors with 32 mm openings and with 60 mm openings, both in normal and lightweight concrete of different strength characteristics and with different transverse reinforcement, were carried out. The experimental study also dealt with the connector height and parallel arrangement of two connectors and their influence on shear resistance. While extensive tests with static loading were carried out, fatigue tests under repeated loading are still in progress. After statistical evaluation of the experimental results and comparisons with other available data the authors developed reasonable shear resistance formulas for all proposed arrangements.

Key words: characteristic resistance; composite steel and concrete structure; design resistance; lightweight concrete; perforated shear connector; push test; shear connector; slip; statistical evaluation.

1. Introduction

Several types of economical and effective connectors are commonly used for composite steel and concrete beams at present (Fig. 1).

Aside from automatically timed welded studs these up-to-date connectors have been developed during last two decades and have become popular due to their advantageous properties. While various out-of-date welded block connectors, anchors, hoops and angles are both laborious and expensive, headed studs have become the most popular shear connectors since early seventieth of last century and major contractors are fully equipped with the necessary automatically timed stud welding equipment. A drawback of the technology is procedure of welding alone, where certain conditions have to be fulfilled (temperature above -18^{o}C, however with caution under 10^{o}C, the clean surface cannot be exposed to falling rain or snow) and need for strong source of direct current straight polarity power.

Therefore, global economy of the connection enforced use of another connectors in some cases. First Hilti brackets fixed by two powder-actuated fasteners were developed (Hilti AG 1984), Fig. 1. This technology requires portable automatic installation equipment only. Moreover a fundamental advantage of all nailed connectors is high quality of the connection independent of moisture on site or of base material coatings, i.e., weather condition and base material surface condition negligibly affect the resulting resistance. However, the installation cost of the brackets in Central Europe is significantly higher in comparison with welded studs and therefore this technology is efficient prevailingly for small

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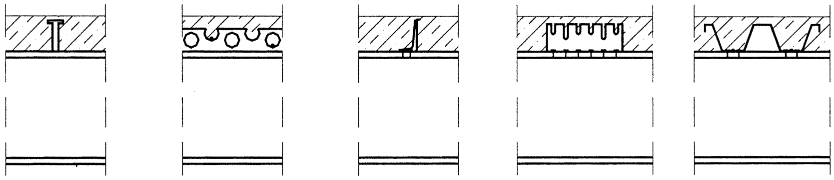

Fig. 1 Currently used economical shear connectors: Headed stud, perforated connector, Hilti bracket, Hilti Ribcon, Hilti Stripcon

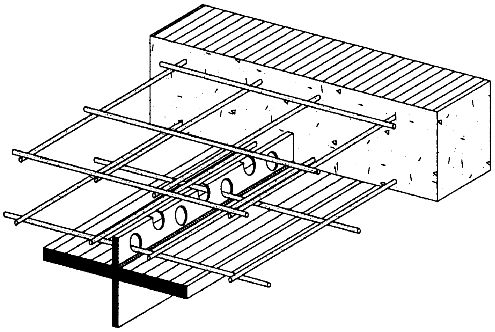

Fig. 2 Perforated shear connector with transverse reinforcement

sites, where furnishing with strong power source needed for welded studs is uneconomical.

Another connecting element called perforated shear connector (or “perfobond”) was developed in the late eighties of last century (Leonhardt *et al.* 1987). Fig. 2 shows how perforated connector is welded by fillet welds (continuous or intermittent) to an upper flange of a girder. Concrete “dowels” going through the perforation, together with a transverse steel reinforcement, provide resistance to shear and uplift forces.

The shear resistance of a perforated shear connector is generally high irrespective of shape of openings. Extensive research of the connector has been carried out (Leonhardt *et al.* 1987, Andrä 1990, Oguejiofor and Hosain 1994, Kraus and Würzer 1997, Ferreira *et al.* 1998, Machacek 1997, Studnicka *et al.* 1999, Rovnak *et al.* 2000, and others). Simplicity, robustness and high shear resistance predetermines applicability of these connectors also for girders with large spans and, due to reasonable fatigue behaviour, for bridge engineering too.

Recently new shear connectors for Hilti Corporation were designed (Fontana and Beck 2000). The thin-walled connectors are fastened to beam flange by powder actuated fasteners and are called Ribcon and Stripcon, see Fig. 1. Extensive research at ETH Zürich led to optimum shapes of these novel connectors.

The principle of the Ribcon shear connector is based on perforated shear connector mentioned above and consists of a thin steel angle with unequal sides cold formed from 1.5-3 mm sheet, whose larger free leg is supplied with various perforations enabling penetrating of concrete. Prescribed number of Hilti powder actuated nails fastens the shorter angle leg to a beam flange. The Ribcon connectors can be fired even through a formwork of trapezoidal sheeting, with waves of sheeting running parallel to the beam. The Ribcon angles may equivalently be replaced by thin-walled channel (U profile) of the same thickness.

Stripcon shear connector follows an idea from early ninetieth of last century (Shanit *et al.* 1991). The connector is suitable especially for use with metal decking whose waves run perpendiculary to axis of the beam. The connector is made of cold formed steel strip of 80 mm width, with a shape corresponding to the wave of the trapezoidal sheeting used as a formwork, the wave of the connector being however higher. Holes are cut in the connector for penetration of concrete and the connector is fastened to a beam by fired nails in its each valley.

Both newly developed connectors (Ribcon and Stripcon) were, in addition and for confirmation of ETH results, tested by Authors in accordance with ENV 1994-1-1 (Eurocode 4). The resulting resistance was published and the connectors recommended for product certification in Czech Republic (Studnicka *et al.* 2001).

Further effort to find another new efficient shear connectors was expended, concerning e.g., oscillating perfobond strip, waveform strip, etc. Galjaard *et al.* (2001). However, their use in practice is limited at present. On the other side an idea to use concrete with steel fibres seems to be useful both for greater strength and ductility of all connectors types.

In the following part the results of research on perforated shear connector undertaken by Authors during last years are presented.

2. Perforated shear connector

Some possible shapes of openings are outlined in Fig. 3 (Leonhardt *et al.* 1987, Institut für Bautechnik 1991, Kraus and Würzer 1997, Rovnak *et al.* 2000).

Two following basic types of the perforated connector were proposed and investigated in CTU Prague:

- connector with 32 mm circular openings;
- connector with 60 mm circular openings.

While the former connector is commonly used for floor structures, the latter is intended especially for use in highway bridges. As railway bridges are concerned, behavior of the connector under repeated loading has to be investigated (fatigue tests in CTU are in progress).

The results of push tests and proposed design resistance for static loading of both connectors are presented in the following chapters.

2.1. Perforated shear connector with 32 mm openings

2.1.1. Push tests

Connector 50/10 [mm] with 32 mm openings (Fig. 4) proposed by the Authors in 1994 was extensively investigated in CTU Prague in normal weight concrete.

Standard push tests were carried out in accordance with Eurocode 4. According to recommendation

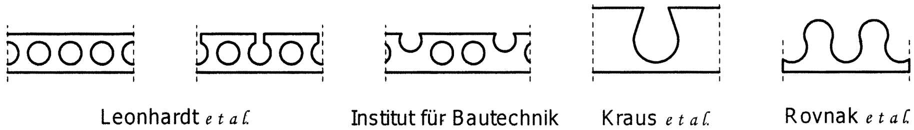

Fig. 3 Various shapes of openings

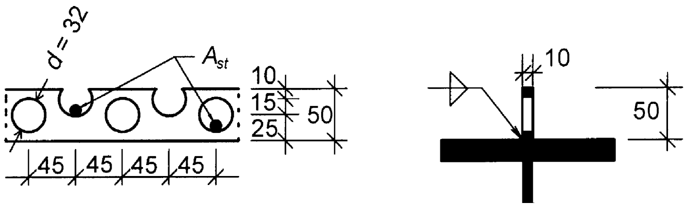

Fig. 4 Shear connector investigated in CTU Prague

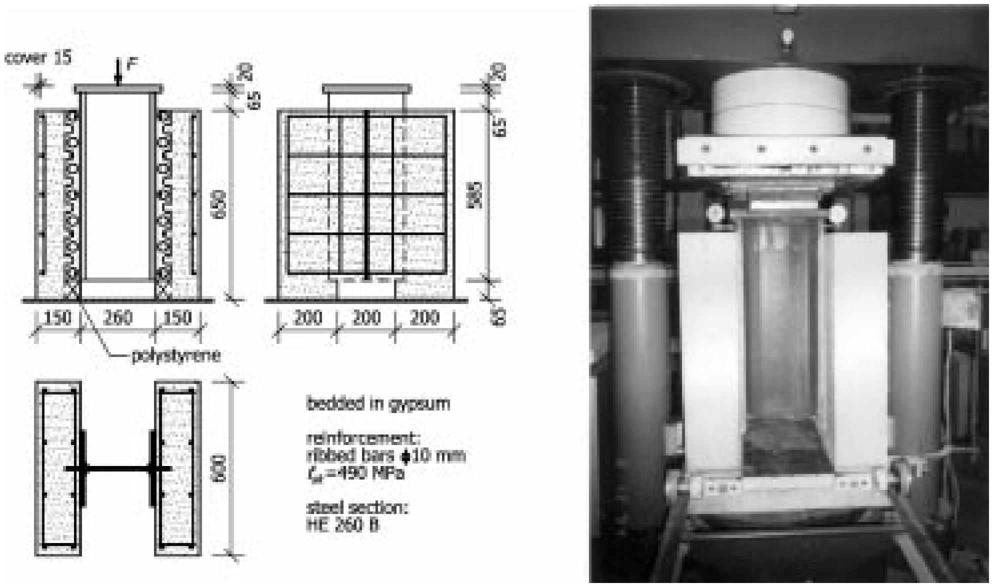

Fig. 5 Push test set up

of this European Prestandard the concrete slab was reinforced by standard steel mesh reinforcement, Fig. 5. Another reinforcement with steel bars of area A_{st} was inserted into openings of the shear connector, Fig. 6. The pouring and compacting of concrete followed the prescribed procedure. Greasing prevented bond at the interface between flanges of steel beam and concrete. Polystyrene blocks to exclude any other support surrounded ends of the shear connectors. Concrete was air-cured and its compressive cylindrical strength investigated in time of specimen push testing.

2.1.2. Test results

In the first period 27 push tests of “basic” 50/10 connector with 32 mm openings were carried out. In the second period modified connectors described later were investigated and, as comparative samples, another four “basic” connectors were tested. The specimens differ in concrete resistance and amount of transverse reinforcement inserted into openings of perforated shear connector.

Some typical load-slip diagrams for various amount of transverse reinforcement are presented for illustration in Fig. 7. The specific tests proved no difference in resistance when the transverse reinforcement was inserted either into “open” or “closed” holes of the connector.

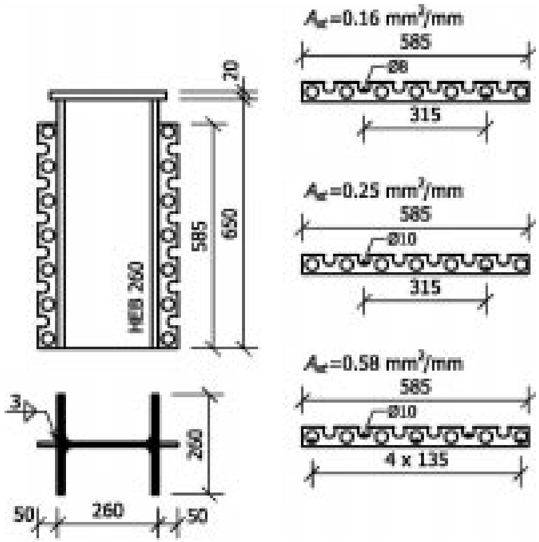

Fig. 6 Steel part of push specimens with typical reinforcement bars

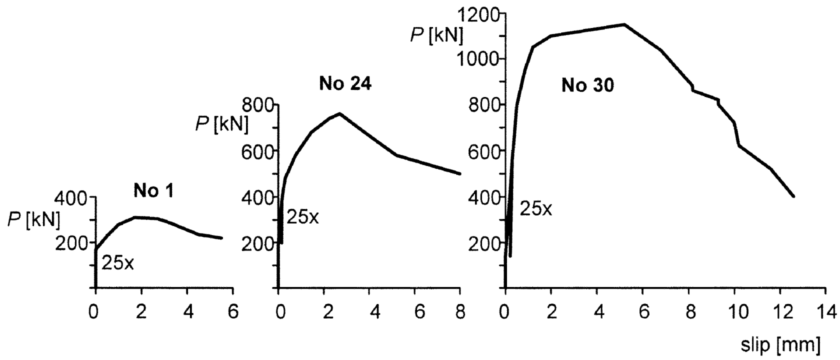

Fig. 7 Typical examples of load-slip diagrams (See Table 1: No. 1 A_{st} = 0.00 mm^2/mm; No. 24 A_{st} = 0.25 mm^2/mm; No. 30 A_{st} = 0.58 mm^2/mm)

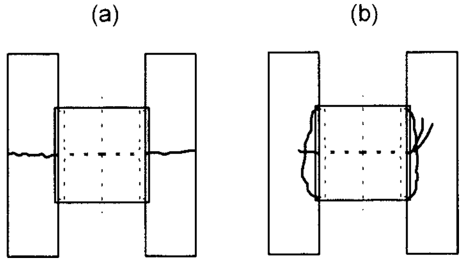

Fig. 8 Typical failure modes of specimen with reinforcement: (a) up to A_{st} = 0.25 mm^2/mm, (b) A_{st} = 0.58 mm^2/mm

Failure modes:

Shear failure of specimens with small transverse reinforcement (up to A_{st} = 0.25 mm^2/mm) was governed by shear splitting of concrete along connector depth line. For bigger transverse reinforcement

(A_{st} = 0.58 mm^2/mm) the behaviour was much more ductile: at first individual reinforcement bars broke down (as can be seen from unloading part of the load-slip diagram) followed by shear collapse at larger volume of concrete decks, see Fig. 8. At some specimens with large transverse reinforcement the connector also broke in its end hole.

The test evaluation for each group of 3 identical tests may be performed simply in accordance with the above-mentioned Eurocode 4 or in accordance with Annex Z of ENV 1993-1-1 (Eurocode 3). The latter, i.e., statistical evaluation procedure was used to receive the characteristic and design resistance of the connector. The procedure started with determination of analytical formula for experimental resistance received through regression analysis of test results. Two independent parameters ($f_{c,\,cube}$, A_{st}) were used and the resistance P_t (see Fig. 9) resulted to:

$$P_t = -87.374 + 12.669 f_{c,cube} + 1020.471 A_{st} \text{ [N/mm]} \tag{1}$$

where

A_{st} [mm^2/mm] is area of transverse reinforcement (steel with characteristic yield point at least f_{sk} = 410 MPa) inserted into openings of the connector (both open and closed holes are taken into account);
$f_{c,cube}$ [MPa] cube concrete strength.

In accordance with Annex Z of Eurocode 3 the coefficient of resistance variation was determined as V_δ = 0.123 and the variation coefficients for basic variables were assumed (as expected in practice) as $V_{fc,cube}$ = 0.12 and $V_{A,st}$ = 0.04. The limited number of test results from the first period (N = 27) was taken into account and factor accounting for introducing the nominal strength of concrete as its characteristic value was determined (k_c = 1.105). The characteristic resistance corresponds to 5% fractile and design resistance to safety index β = 3.8 with corresponding fractile factor for 27 tests $u_{d,n}$ = 3,43. The procedure gave eventually (Machacek 1997):

characteristic resistance:

$$P_{Rk} = -68 + 12.4 f_{ck} + 797 A_{st} \text{ [N/mm]} \tag{2}$$

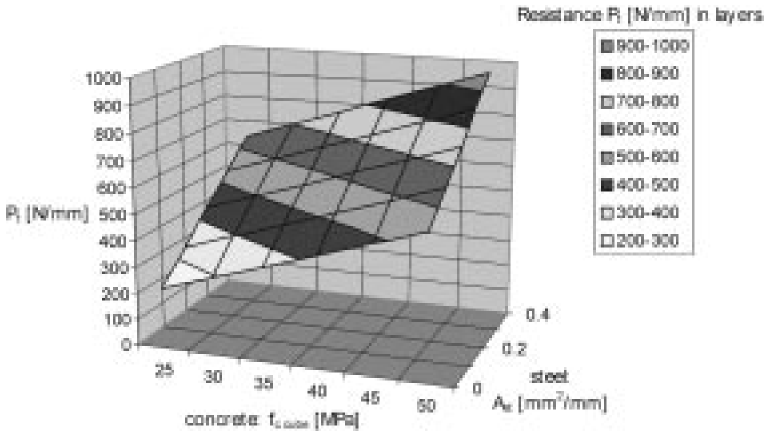

Fig. 9 Result of regression analysis

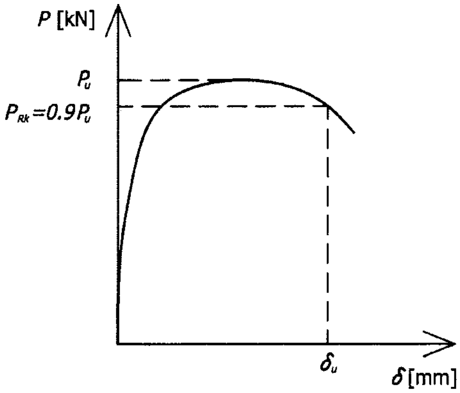

Fig. 10 Determination of slip capacity δ_u

design resistance:

$$P_{Rd} = -49 + 8.8f_{ck} + 568A_{st} \text{ [N/mm]} \tag{3}$$

where

f_{ck} [MPa] is characteristic (cylindrical) concrete strength.

The corresponding partial safety factor ($\gamma_v = P_{Rd} / P_{Rk}$):

$$\gamma_v = 1.40 \tag{4}$$

Besides the resistance the slip capacity of the connector is also important feature for its practical use. In accordance with Eurocode 4 any connector may be regarded to be ductile provided its characteristic slip is not less than 6 mm. In consequence such ductile connector enable to justify the assumption of ideal plastic behaviour of the shear connection. The slip capacity δ_u should be taken as the maximum slip measured at the characteristic load level, see Fig. 10. The characteristic slip δ_{uk} should be taken as the minimum test value of δ_u reduced by 10% or determined by statistical evaluation.

Resulting experimental shear strengths P_{exp} [N/mm] of all 31 push tests of "basic" connector with different amounts of transverse reinforcement A_{st} are presented in Table 1. Shear values in the last but one two columns of the table are calculated according to following formulas:

a) Average shear strength P_A based on regression analysis of the former 27 tests results, using Eq. (1) after replacing $f_{c,cube}$ for $f_{c,cyl}$ (Machacek and Studnicka 1997):

$$P_A = -87.374 + 15.836\, f_{c,cyl} + 1020.471\, A_{st} \tag{5}$$

b) Characteristic shear resistance P_{Rk} according to Eq. (2), calculated for $f_{ck} = f_{c,cyl}$:

$$P_{Rk} = -68 + 12.4\, f_{c,cyl} + 797A_{st} \text{ [N/mm]} \tag{6}$$

An attempt to use another formulas for shear resistance of perforated connector was made. First formula according to Oguejiofor and Hosain (1994) was analyzed (average shear strength based on regression analysis of samples with 13 mm thick connector and 50 mm openings). However, the thickness and finite length of the connector with transfer of forces at the connector head (including the influence of transverse reinforcement at this location) are far from the CTU Prague connector

Table 1 Test results of “basic“ 50/10 connector with 32 mm openings

No	Reinforcement (f_{sk} = 490 MPa) A_{st} [mm^2/mm]	Concrete $f_{c,cyl}$ [MPa]	P_{exp} [N/mm]	P_A accord. (5) [N/mm]	P_{Rk} accord. (6) [N/mm]	δ_u [mm]
1	0	20.0	263	229	180	3.6
2	0	20.0	250	229	180	2.3
3	0	23.5	282	285	223	3.3
4	0	24.8	308	277	240	3.8
5	0	24.8	310	277	240	2.3
6	0	24.8	313	277	240	2.2
7	0	24.8	299	277	240	3.4
8	0	24.8	276	277	240	1.9
9	0	24.8	333	277	240	1.3
10	0	30.9	387	373	315	3.1
11	0	30.9	333	373	315	5.5
12	0	30.9	351	373	315	3.8
13	0.16	23.7	550	451	353	2.0
14	0.16	32.4	593	599	461	4.9
15	0.16	37.6	568	671	526	4.1
16	0.25	19.0	427	469	367	2.5
17	0.25	19.0	438	469	367	2.2
18	0.25	19.0	470	469	367	2.8
19	0.25	28.6	740	621	486	5.2
20	0.25	35.5	825	730	571	4.2
21	0.25	35.5	855	730	571	4.4
22	0.25	35.5	829	730	571	5.3
23	0.25	36.0	667	738	578	3.9
24	0.25	36.0	650	738	578	3.6
25	0.25	36.0	654	738	578	2.0
26	0.35	32.5	784	784	614	6.0
27	0.58	18.8	816	802	627	6.5
28	0.58	28.6	927	957	749	7.1
29	0.58	32.6	1103	1021	799	7.1
30	0.58	32.6	983	1021	799	6.8
31	0.58	32.6	946	1021	799	6.5

arrangement and the formula cannot be used.

Second formula in accordance with German license (Institut für Bautechnik 1991) can also not be used as the shear resistance there depends on concrete strength only and the amount of transverse reinforcement is not precisely covered in the formulation.

All characteristic values P_{Rk} in Table 1 are well under experimental values and the statistical procedure used seems to be adequate. As slips are concerned, the values of slip capacity are dependent on amount of transverse reinforcement. An estimate was made to consider perforated connector with at

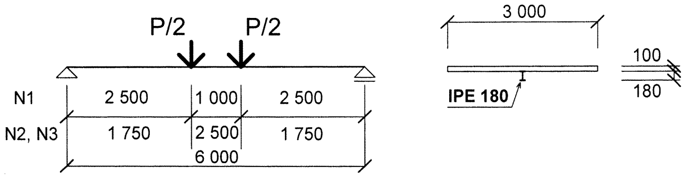

Fig. 11 Geometry of tested composite girder

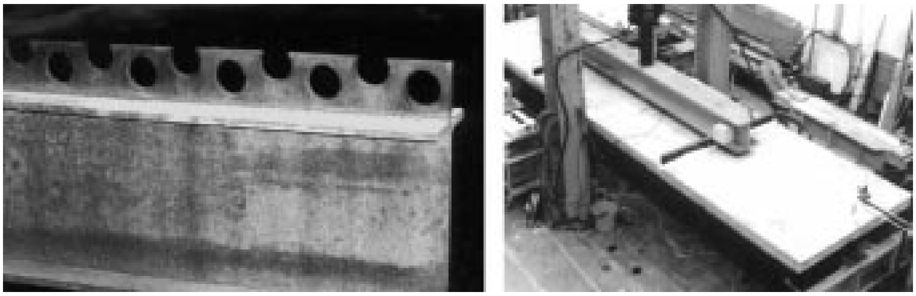

Fig. 12 Girder with perforated shear connector (connector detail on the left, testing on the right)

least A_{st} = 0.5 mm^2/mm as ductile while lesser transverse reinforcement results in non-ductile connector in accordance with Eurocode 4. In general the initial slip (say up to a half of the P_{exp}) may be considered very low (conservatively 0.2 mm).

2.1.3. Girder tests

The push test results were verified experimentally on three tests with composite girders (N1, N2, N3) of reasonable size (span 6 m). Girders N1 and N3 were designed with full shear connection while N2 with partial shear connection (Machacek and Studnicka 1999), Fig. 11, Fig. 12.

Comparison of experimental and theoretical values is given in Table 2. Experimental resistance of shear connectors P_{exp} corresponds to values of push tests performed with the same connectors in time of the girder test and therefore differs slightly from values according to Eq. (5) ($A_{st,\mathrm{N1}}$ = 0.160 mm^2/mm, $A_{st,\mathrm{N2}} = A_{st,\mathrm{N3}}$ = 0.090 mm^2/mm). Theoretical values correspond to simple elastic and plastic analysis or elastic shear connection collapse when taking into account given resistance of perforated connector, all in accordance with Eurocode 4.

Table 2 Theoretical and experimental collapse loads of the tested composite girders

Girder	Concrete		Steel f_y [MPa]	Perforated connector P_{exp} [N/mm]	Collapse loading P [kN]			
	f_{ck} [MPa]	E_{cm} [GPa]			Theoretical values			Experimental
					Elastic theory	Plastic theory	Shear connection	values
N1	28.7	33.0	282.1	436.0	45.7	83.8	141.2	93.2
N2	12.8	26.3	282.1	314.9	62.1	110.1	92.2	105.0
N3	16.3	27.6	282.1	398.8	63.0	113.9	124.3	114.0

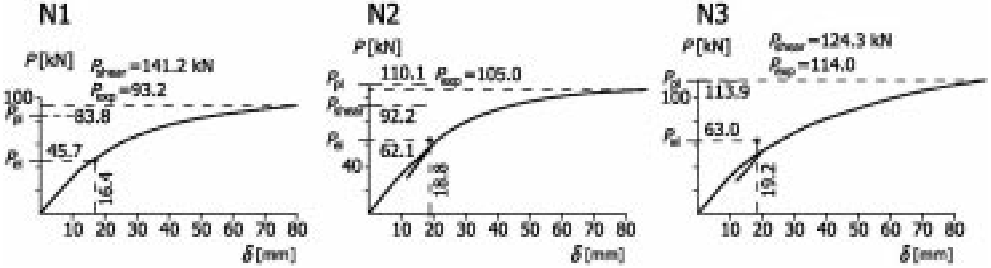

Fig. 13 Load-deflection curves of tested girders at mid-span

The experiments confirmed the expected behaviour of shear connection. Deflections and strains of all the three girders in elastic region proved to be well in accordance with calculated values. After reaching experimental values given in Table 2 the tests finished with enormous deflections in mid-span, approaching 200 - 250 mm. No shear splitting around shear connectors was observed at collapse when finishing the tests. End slip between steel girder and concrete deck at the above given collapse for girder N1 was negligible, for girder N2 only 0.2 mm and for N3 reached 1.74 mm. However, the slips near collapse were highly non-linear and therefore the values are illustrative only. Experimental collapse load of girder N2 with partial shear connection approached the theoretical plastic value. It means that reasonable plastic redistribution of shear flow took part for this perforated shear connector. Deflection curves of all the three girders are presented in Fig. 13 (for more details see Machacek and Studnicka 1999).

Nevertheless, the authors recommend the use of elastic theory for practical design of beams with this type of shear connection, because ductility of the connector with its characteristic slip δ_u = 1.3 - 7.1 mm does not correspond fully to value recommended by Eurocode 4, i.e., δ_{uk} = 6 mm. Ductile behaviour of the connectors can be expected and use of plastic design may be adopted only for large transverse reinforcement (recommended value A_{st} > 0.5 mm^2/mm).

2.1.4. Modified connector with 32 mm openings

Recently modified perforated connector has been proposed for thicker concrete slabs, Fig. 14. The connector 100/10 [mm] has openings situated in its upper part only and is intended for use together with thin concrete precast deck (used as a formwork) or with thicker concrete decks.

Shear resistance of the modified connectors was supposed to be described by simple coefficient (expressing a percentage difference) in respect to the single "basic" 50/10 connector. Therefore, each

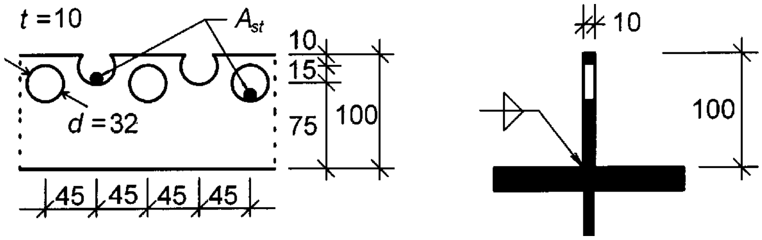

Fig. 14 Modified perforated connector with 32 mm openings

Table 3 Test results for “basic“ 50/10 and modified 100/10 connectors

No	Connector	Reinforcement (f_{sk} = 490 MPa) A_{st} [mm^2/mm]	Concrete $f_{c,cyl}$ [MPa]	P_{exp} [N/mm]	Ratio	δ_u [mm]
	“basic”	0.16	37.6	568		4.1
1	100/10	0.16	37.6	700	1.23	2.7
	“basic”	0.35	32.5	784		6.0
2	100/10	0.35	32.5	898	1.15	4.8

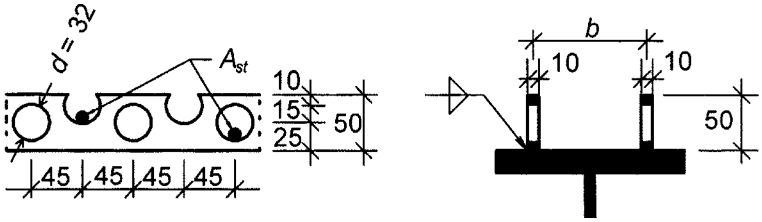

Fig. 15 Double arrangement of connectors

Table 4 Test results for “basic“ 50/10 single and double connectors

No	Connector	b Fig. 15 [mm]	Reinforcement (f_{sk} = 490 MPa) A_{st} [mm^2/mm]	Concrete $f_{c,cyl}$ [MPa]	P_{exp} [N/mm]	Ratio	δ_u [mm]
	single	-	0.16	32.4	593		4.9
1	double	100	0.16	32.4	983	1.66	4.1
	single	-	0.35	32.5	784		6.0
2	double	100	0.35	32.5	1314	1.68	4.6
	single	-	0.16	23.7	550		2.0
3	double	125	0.16	23.7	932	1.69	2.8
	single	-	0.35	22.5	784		6.4
4	double	125	0.35	22.5	1144	1.46	4.0
	single	-	0.16	23.7	550		2.0
5	double	150	0.16	23.7	1017	1.85	4.0
	single	-	0.35	22.5	784		6.4
6	double	150	0.35	22.5	1101	1.40	4.0

series of 100/10 size arrangement included also a comparative push test with “basic” single 50/10 connector. The results are summarized in Table 3.

Another modification concerns double (parallel) arrangement of the "basic" connector, Fig. 15. Such an arrangement is useful to cover high shear forces in primary beams and similar structures.

The test results are presented in Table 4. Again, each series of double connector arrangement included a comparative push test with “basic” single 50/10 connector.

The following recommendations can be proposed after evaluating all presented experimental results:

1) Characteristic and design resistance of the “basic” 50/10 connector with 32 mm openings may be taken in accordance with (2) and (3) respectively and partial safety factor according to (4).

2) The 50/10 connector with 32 mm openings may be considered as ductile in accordance with

Eurocode 4 provided the amount of transverse reinforcement $A_{st} > 0.5$ mm^2/mm.

3) Resistance of "high" connector 100/10 with 32 mm openings in its upper part may be considered as the one for "basic" connector increased by 10%. Such value covers safely test results for both tested amounts of transverse reinforcement.

4) Slip values for "high" connector are significantly lower in comparison with "basic" one. The "high" connector cannot be considered as ductile.

5) Resistance of "basic" connectors in double (parallel) arrangement in mutual distance $b = 100$ [mm] in accordance with Fig. 15 may be considered as the one for "basic" connector increased by 40%. This estimate represents safely test results for transverse reinforcement with $A_{st} \geq 0.16$ mm^2/mm up to $A_{st} = 0.6$ mm^2/mm and the three different distances of connectors. As seen from Table 4 the value ranges from 40% to 85% in rather illogical manner and more tests are needed to cover safely the behaviour (such investigation is in progress).

6) Slip values for parallel arrangement of connectors are comparable with slips of single "basic" one and the same recommendation may be done for design (ductile behaviour for $A_{st} > 0.5$ mm^2/mm).

2.2. Perforated shear connector with 60 mm openings

2.2.1. Experimental program

The connector 100/12 having 60 mm openings is supposed to be used for bridge girders with thicker concrete decks, Fig. 16. The resistance of the connector was tested both in normal weight and lightweight concrete. Push specimens were arranged in similar way as shown in Fig. 5 and Fig. 6 with length of the perforating connectors being 630 mm.

Load-slip diagrams received from tests for some of these connectors are shown in Fig. 17.

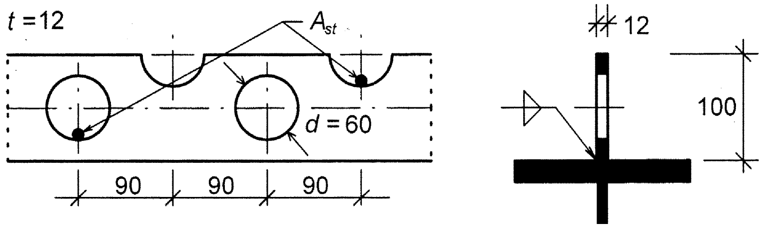

Fig. 16 Connector 100/12 with 60 mm openings investigated in CTU

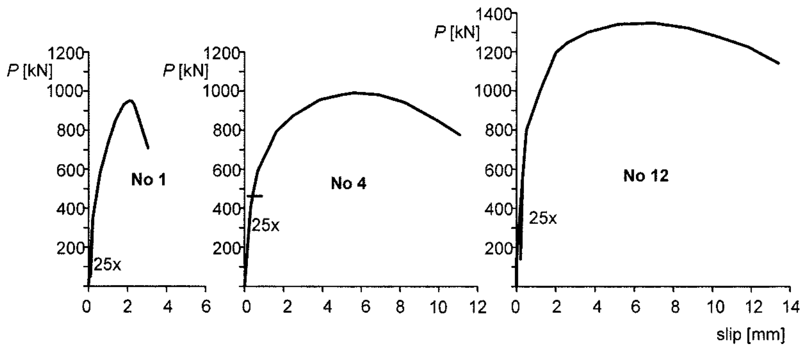

Fig. 17 Typical load-slip diagrams for connector with 60 mm openings in normal weight concrete (See Table 5: No. 1 $A_{st} = 0.00$ mm^2/mm; No. 4 $A_{st} = 0.25$ mm^2/mm; No. 12 $A_{st} = 0.50$ mm^2/mm)

Table 5 Test results for 100/12 connector with 60 mm openings

No	Reinforcement (f_{sk} = 490 MPa) A_{st} [mm^2/mm]	Concrete $f_{c,cyl}$ [MPa]	P_{exp} [N/mm]	P_{Rk} accord. (7) [N/mm]	δ_u [mm]
1	0	30.1	754	698	2.6
2	0	30.1	754	698	3.6
3	0	30.4	754	702	4.9
4	0.25	23.1	790	677	9.0
5	0.25	23.1	794	677	9.1
6	0.25	23.1	825	677	9.0
7	0.25	30.4	913	780	10.6
8	0.50	27.2	1127	813	6.5
9	0.50	27.2	1167	813	6.6
10	0.50	27.2	1032	813	6.3
11	0.50	38.0	1048	965	10.2
12	0.50	38.0	1071	965	10.9
13	0.50	38.0	1032	965	10.1
14	0.72	30.4	1190	927	9.0
15	1.28	22.6	1040	992	9.5
16	1.28	22.6	1111	992	10.0

Results of 16 push tests covering various transverse reinforcements A_{st} (in all tests with diameter 10 mm) are presented in Table 5. Instead of statistical approach (using Annex Z of Eurocode 3) the simplified procedure suggested by Eurocode 4 for evaluation of push test results was used and characteristic resistance of each test determined. Regression analysis resulted in following formula for characteristic resistance:

$$P_{Rk} = 273 + 14.1\, f_{ck} + 313\, A_{st} \text{ [N/mm]} \tag{7}$$

The last but one column of Table 5 presents characteristic values received from Eq. (7), using $f_{ck} = f_{c,cyl}$.

All characteristic values P_{Rk} in Table 5 are well under experimental values and the Eq. (7) seems to be adequate. As slips are concerned, the values of slip capacity are dependent on amount of transverse reinforcement and strength of concrete. It is proposed to consider perforated connector with at least A_{st} = 0.25 mm^2/mm and in concrete with strength over 20 MPa as ductile otherwise as non-ductile in accordance with Eurocode 4.

2.2.2. Connector with 60 mm openings in lightweight concrete

For lightweight concrete original Czech extruded aggregate Liapor® was used. Unit mass of tested lightweight concrete was ρ = 1600 - 1770 [kg/m^3] and its cylindrical strength within 20 and 40 MPa.

Results of 9 push tests with 100/12 connector and 60 mm openings used for lightweight concrete are presented in Table 6.

Slips δ_u were between 3.0 – 9.1 mm (more ductile behaviour again for larger values of A_{st} and for concrete with higher strength).

Analyzing the results a simple relation between shear resistance in normal weight and lightweight concrete was found corresponding to Eurocode 4 reduction for tensile strength of lightweight concrete.

Table 6 Test results for 100/12 connector with 60 mm openings in lightweight concrete

No	Reinforcement (f_{sk} = 490 MPa) A_{st} [mm^2/mm]	Concrete $f_{c,cyl}$ [MPa]	P_{exp} [N/mm]	P_{Rk} accord. (7), (8) [N/mm]	δ_u [mm]
1	0	20.5	389	343	3.0
2	0	23.5	397	369	4.2
3	0	30.6	595	480	4.4
4	0.25	20.5	437	391	4.2
5	0.25	23.5	516	417	4.7
6	0.25	30.6	619	533	5.0
7	0.72	20.5	556	481	6.5
8	0.72	23.5	615	507	6.5
9	0.72	30.6	762	633	9.1

Therefore, the shear resistance of connector in lightweight concrete is given by formula (7) multiplied by factor η:

$$\eta = 0.3 + 0.7\left(\frac{\rho}{2400}\right)^2 \tag{8}$$

where

ρ [kg/m^3] is unit mass of lightweight concrete.

In the last but one column of Table 6 the characteristic resistance of the connector calculated using Eqs (7) and (8) is presented.

The following recommendations can be proposed after evaluating all presented experimental results:

1. Characteristic resistance of 100/12 connector with 60 mm openings may be taken in accordance with Eq. (7). Corresponding partial safety factor according to Eurocode 4 is γ_v = 1.25.

2. Characteristic resistance of 100/12 connector with 60 mm openings in lightweight concrete may be obtained from Eq. (7) multiplied by reduction coefficient according to (8). Nevertheless, more tests are needed for better understanding of the connector behaviour in lightweight concrete.

3. Slip of the 100/12 connector with 60 mm openings in normal concrete and transverse reinforcement $A_{st} \geq 0.25$ mm^2/mm fulfils requirement $\delta_{uk} \geq 6$ mm requested for ductile connectors by Eurocode 4. For lightweight concrete the amount of transverse reinforcement for ductile connectors should be $A_{st} \geq 0.70$ mm^2/mm unless additional tests results are available.

3. Conclusions

Two types of perforated shear connector were tested (first with 32 mm and second with 60 mm openings) for shear capacity. Test results and their statistical evaluation enabled to determine average, characteristic and design shear resistance for limit states design according to Eurocode 4. Important feature of the resistance formulas is dependence on the amount of transverse reinforcement inserted into the openings, while both “open” and “closed” openings may be considered as equivalent ones. Amount of transverse reinforcement may change along the length of a girder significantly, enabling more economic design (Machacek and Studnicka 1999). The reinforcement is usually present as necessary reinforcement of the concrete slab (otherwise additional reinforcement of its double anchor length is used). The proposed values for connector with 32 mm openings were successfully verified in tests of three real size girders.

Connector with 32 mm openings was also tested for two other modifications (the first one with another position of the openings in the connector profile, the second with parallel arrangement of the two identical connectors). The tentative design formulas were proposed to cover shear resistance for both modifications.

The connector with 60 mm openings was tested for use both in normal weight and in lightweight concrete and design formulas for shear resistance were proposed.

Classification of the connectors concerning their ductility was proposed in accordance with Eurocode 4.

Presented design recommendations are valid for static loading only, while fatigue tests are still in progress.

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