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Universidade do Minho

Research Report

Fatigue strength of shear connectors

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LIST OF SYBOLS

f_c - cylinder strength of the concrete (MPa)
f_y - yield strength of shear connector material (MPa)
f_u - tensile strength of shear connector material (MPa)
d - diameter of steel stud connector (mm)
L - length of channel connector or Perfobond connector (mm)
P_{max} - the maximum load per connector(kN)
P_{min} - the minimum load per connector(kN)
ΔP - P_{max}-P_{min}, load range (kN)
$\Delta\tau$ - ΔP /A(A-shear area of connector), stress range(MPa)
N - number of cycles
N_e - experimental fatigue life
N_f - theoretical fatigue life
Δq - stress range of channel connector, N/mm of one channel
P_u - ultimate strength per connector due to static loading (kN)
P_s - residual static strength per connector after certain cycling numbers (kN)
s_{max} - the maximum slip per cycle (mm)
s_{min} - the minimum slip per cycle (mm)
δ - slip per cycle (mm)
m, k - exponents in expression for fatigue life
K_1 K_2 - coefficients

1. Introduction

1.1. Types of shear connectors

Shear connection is essential for steel-concrete composite structures, as shown in Figure 1, only when the interface between steel beam and concrete slab is eliminated, the two components can act together as a composite unit.

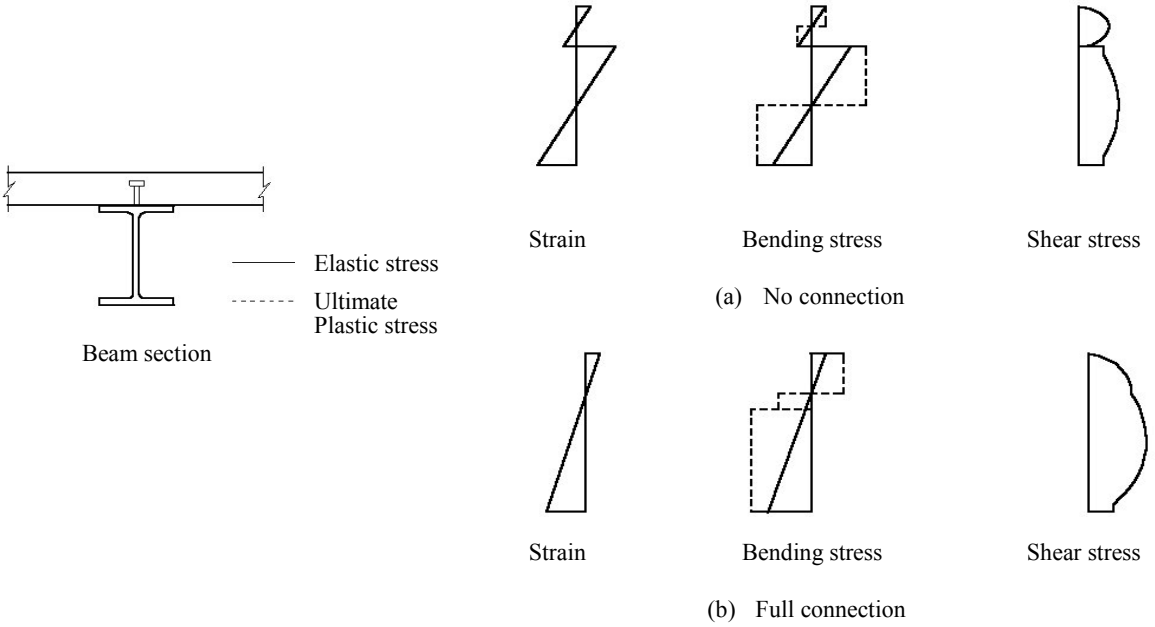
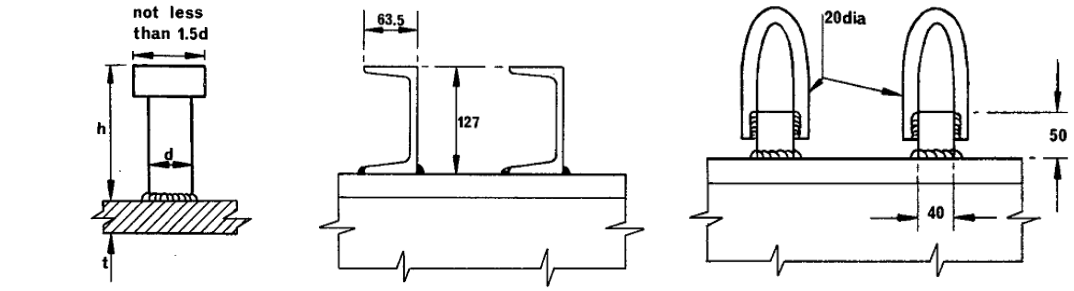


Figure 1 - Effect of shear connection on strain, bending and shear stresses

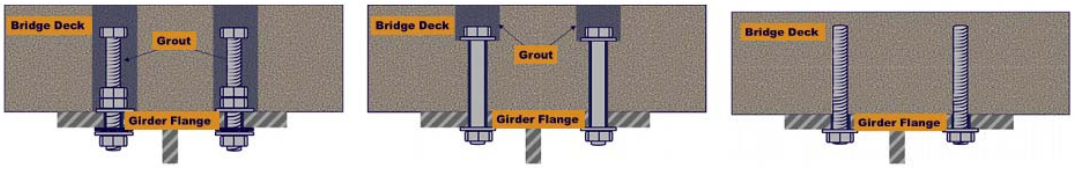
Since 1940s, several types of shear connectors have been proposed and used in composite structures (Figure 2), such as headed stud connector, channel connector, block connector with hoops, post-installed shear connector, T connector, Perfobond, T-Perfobond and Crestbond (CR connector).

The most well-known and used connector is the headed stud, developed during the 40's by Melson Stud Welding Company. These connectors are industrially produced and are available in various diameters, usually varying from 6 to 23 mm, and various heights, that can go from 30 to 500 mm. The main advantage of this connector are as the stud welding is fast, it anchors well in concrete and it is easy to dispose the reinforcement through the slab, between the studs. Some disadvantage can also be pointed out as it needs high energetic resources to work, fatigue problems can also occur for service load level.

Since the maximum shear force that can be resisted by a stud is relatively low, other types of connector with higher strength have been developed in the early times, primarily for use in bridges as block connector with hoops and channel connector.



(a) Headed stud (b) Channel (c) block with hoops



(d) Post-installed shear connectors (Gunup, 2010)

(e) T (Valente, 2007)

(f) Perfobond (Valente, 2007)

(g) T-Perfobond (Vianna, 2008)

(h) Crestbond (Veríssimo, 2006)

Figure 2 - Several types of connectors

The post-installed shear connectors are used to develop the composite action in existing non-composite floor systems, in a number of older bridges built before the 1970's.

The T connector can be produced with different shapes, but usually is made from a commercial profiled steel section. The T shape has a larger contact area than a single strip, so the shape is appropriate to prevent vertical separation between the steel beam and the concrete slab. In terms of fatigue, T connectors show the same fatigue problems as studs, as they present important deformation for service loadings, and they need to develop high deformation to mobilize the

maximum load.

The Perfobond rib shear connector was developed by a German office in the late 1980's, moved by the unsatisfactory behavior of shear studs that result from fatigue problems. Perfobond connectors can be produced in large scale with different shapes and sizes, they can easily be welded without special equipment at site or at factory, and a significant number of studs can be replaced by a smaller number of Perfobond ribs, as this connector shows a very high load bearing capacity.

T-Perfobond connector derives from the Perfobond connector by adding a flange to the plate, acting as a blocking. This kind connector could combine the large strength of a block type connector with some ductility and uplift resistance arising from the holes at the Perfobond connector web. Because of combination, it is complicated to calculate the shear strength. It might be applied only in some specific projects.

The principal disadvantage of Perfobond is the difficulty of placing the transversal bottom slab reinforcement. To avoid this disadvantage, some similar improved types of connectors like Crestbond also called CR connector have been invented and studied, considering open apertures on the plates.

1.2. Motivation and objectives

Nowadays steel and concrete composite beams are widely applied in structures mainly subjected to dynamic loads, such as railway and road bridges exposed to traffic loads and industrial crane runways exposed to folk-lift truck loads, the fatigue resistance of these structures must be generally and carefully verified. And normally the shear forces between concrete flange and steel beams are all transferred by different kinds of shear connectors, fatigue failure of composite beam may happen on shear connectors. So the fatigue resistance of shear connectors in composite beams is significant for the safe of whole structure and needed to be well investigated.

The fatigue strength of shear connectors could be explained in three ways, the fatigue endurance described by the relationships between stress range (S) and number of cycles (N), the static residual strength after a certain number of cycling loading, and the slip behavior during the cycling tests. S-N curves which widely used for steel fatigue assessment are also useful for shear connectors. Although there are already S-N curves for headed steel studs in Eurocodes and AASHTO, they are normally available for common studs. Whether these S-N curves applicable for large diameter studs or studs embedded in high strength concrete or other kind shear connectors is questionable. And the factors influencing fatigue life of shear connectors are needed more research. Furthermore, for the fatigue life assessment of existing structures, it is important to attain the residual strength after a certain service life or the principles of slip

developing behavior during the service life. Then it is possible to assess how long the shear connectors could still service.

Based on forenamed reasons, the following report surveys almost all the available test results for fatigue strength of shear connectors around the world, including summaries of the research on endurance, residual strength and slip of shear connectors under cycling loading. The objectives of this report are the following:

- to describe the test specimens and test procedures each author used for fatigue testing;
- to present the test results of endurance, the residual strength and the slip behavior of shear connectors;
- to compare the test results between different authors';
- to discuss the applicable S-N curves for designing shear connectors;
- to evaluate the factors influencing the fatigue life of shear connectors;
- to analyze the slip developing principles during the fatigue testing;
- to generalize methodology for research on fatigue behavior of new kind connectors;

.

2. Survey over endurance tests

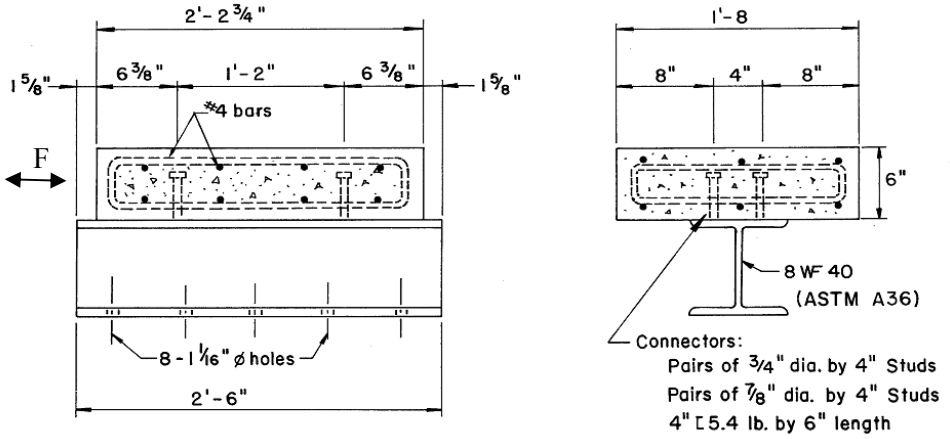
2.1. Tests by Slutter and Fisher

2.1.1. Tests for headed studs

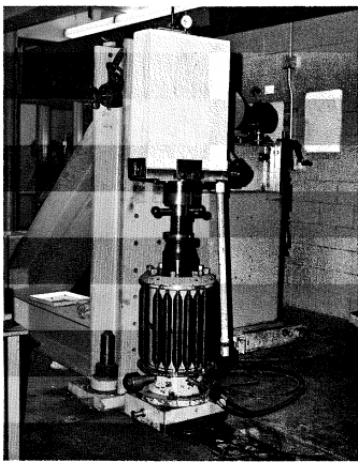
Slutter and Fisher (1966) examined the fatigue resistance on 46 stud connector specimens by beam tests as shown in Figure 3.

All the specimens' slabs had the same geometry, with 680×500×150 mm^3, including the same distribution of #4 bars (13mm in diameter), 4 transversal and 3 longitudinal rebars were disposed. The concrete flange was casted in horizontal position, as in a real structure.

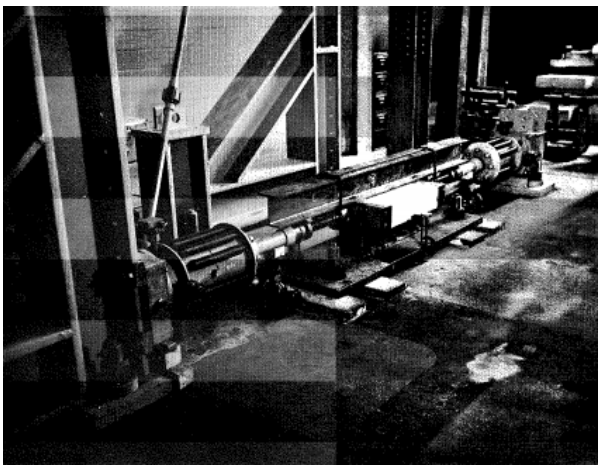
Different stud diameters and various flange thickness were tested. These were the parameters under study on the fatigue resistance of the tested specimens. Therefore, three types of steel section were used: section 8WF40 with flange thickness t_f = 14.3 mm, section 8WF31 with flange thickness t_f = 11.1 mm, and section 8WF67 with flange thickness t_f = 23.8 mm. The test specimens were divided in two groups: one that includes unidirectional loading and other that regards reversal loading. The test details and results are listed up in Table 1. Tests number 1 to 42 are with section 8WF40, number 43 and 44 are with section 8WF31, and number 45 and 46 are with section 8WF67.



(a) Dimension



(b) Unidirectional loading



(c) Reversal loading

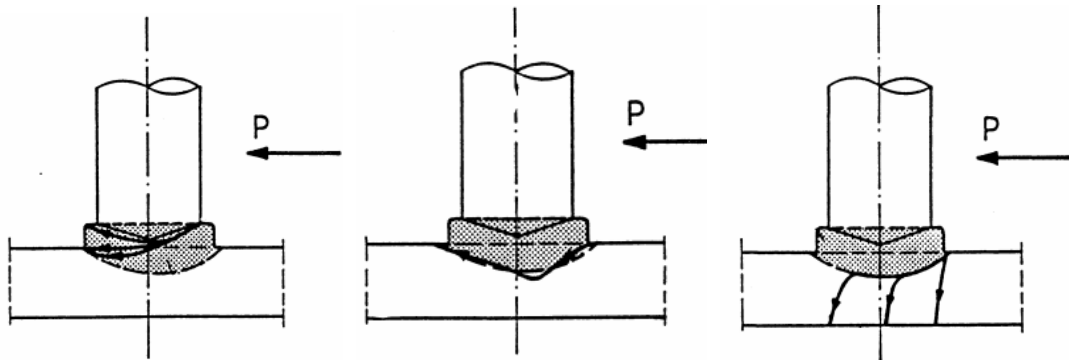
Figure 3 - Push-out-specimen used by Slutter and Fisher (1966)

Table 1 - Test results of stud connectors by Slutter and Fisher (1966)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	a1A	25.9	342.1	423.8	19	19.8	-11.9	31.7	112	1587.4
2	b1A	26.3	342.1	423.8	19	19.8	-11.9	31.7	112	1975.2
3	c1A	26.9	342.1	423.8	19	19.8	-11.9	31.7	112	2557.5
4	a2A	25.8	342.1	423.8	19	27.8	-11.9	39.7	140	104.6
5	b2A	33.2	342.1	423.8	19	27.8	-11.9	39.7	140	104.8
6	c2A	27.4	342.1	423.8	19	27.8	-11.9	39.7	140	171.1
7	a3A	30.8	342.1	423.8	19	35.7	-11.9	47.6	168	106.5
8	b3A	26.4	342.1	423.8	19	35.7	-11.9	47.6	168	85.5
9	c3A	26.9	342.1	423.8	19	35.7	-11.9	47.6	168	30.6
10	a2B	29.5	342.1	423.8	19	27.8	4.0	23.8	84	897.3

11	b2B	30.9	342.1	423.8	19	27.8	4.0	23.8	84	565.3
12	c2B	31.0	342.1	423.8	19	27.8	4.0	23.8	84	551.1
13	a3B	31.5	342.1	423.8	19	35.7	4.0	31.8	112	139.4
14	b3B	31.3	342.1	423.8	19	35.7	4.0	31.8	112	114.7
15	c3B	30.2	342.1	423.8	19	35.7	4.0	31.8	112	199.5
16	a4B	33.7	342.1	423.8	19	43.7	4.0	39.7	140	41.5
17	b4B	30.3	342.1	423.8	19	43.7	4.0	39.7	140	50.7
18	c4B	31.5	342.1	423.8	19	43.7	4.0	39.7	140	58.7
19	a3C	30.8	342.1	423.8	19	35.7	19.8	15.9	56	7481.1
20	b3C	31.1	342.1	423.8	19	35.7	19.8	15.9	56	10276.0
21	c3C	30.5	342.1	423.8	19	35.7	19.8	15.9	56	5091.1
22	a4C	33.0	342.1	423.8	19	43.7	19.8	23.8	84	798.0
23	b4C	30.7	342.1	423.8	19	43.7	19.8	23.8	84	1215.4
24	c4C	30.9	342.1	423.8	19	43.7	19.8	23.8	84	1010.4
25	a5C	31.1	342.1	423.8	19	51.6	19.8	31.8	112	335.8
26	b5C	32.2	342.1	423.8	19	51.6	19.8	31.8	112	99.2
27	c5C	32.6	342.1	423.8	19	51.6	19.8	31.8	112	197.0
28	a6B	22.7	398.2	495.0	19	23.8	4.0	19.8	70	662.5
29	b6B	23.0	398.2	495.0	19	23.8	4.0	19.8	70	919.1
30	c6B	23.6	398.2	495.0	19	23.8	4.0	19.8	70	1144.6
31	a6C	22.2	398.2	495.0	19	39.7	19.8	19.9	70	1213.6
32	b6C	22.2	398.2	495.0	19	39.7	19.8	19.9	70	1295.3
33	c6C	22.2	398.2	495.0	19	39.7	19.8	19.9	70	1618.9
34	e1G	35.1	407.1	450.5	22	19.8	-12.2	31.9	84	1056.4
35	e2G	34.0	407.1	450.5	22	27.7	-12.2	39.9	105	218.6
36	e3G	30.9	407.1	450.5	22	35.7	-12.2	47.9	126	48.3
37	e2H	31.2	407.1	450.5	22	27.7	3.8	24.0	63	2133.0
38	e3H	30.9	407.1	450.5	22	35.7	3.8	31.9	84	112.5
39	e4H	31.4	407.1	450.5	22	43.7	3.8	39.9	105	33.0
40	e3I	31.3	407.1	450.5	22	35.7	19.8	16.0	42	4885.1
41	e4I	32.3	407.1	450.5	22	43.7	19.8	24.0	63	587.7
42	e5I	31.8	407.1	450.5	22	51.7	19.8	31.9	84	134.3
43	8WF31	27.5	342.1	423.8	19	27.8	4.0	23.8	84	405.5
44	8WF31	27.5	342.1	423.8	19	27.8	4.0	23.8	84	727.5
45	8WF67	27.5	262.7	437.3	19	27.8	4.0	23.8	84	205.5
46	8WF67	27.5	262.7	437.3	19	27.8	4.0	23.8	84	125.1

These pioneer tests show several most important ideas for fatigue behaviors of headed studs. First, two types of fatigue cracks occurred. In Type A, the beginning of crack formed at the stud shank and the successive crack forms through the shank and the weld collar respectively; in Type B the beginning of crack formed at the foot of the weld collar and courses the region of welding influence in the flange material. According to these results and the sequent research in this topic, Hallam (1976) summarized three types of fatigue cracks as shown in Figure 4. In the Type C, the beginning of crack formed at the foot of the weld collar and successive crack forms either directly through the flange or first alongside the welding influence zone and then through the flange. This type can usually be observed in beams with high tensile stress in the flange and simultaneously high shear stresses in the interface.



Type A Type B Type C

Figure 4 - Failure models of Shear studs under fatigue loading (Hallam, 1976)

Second, the stress range instead of peak load is the main influencing factor for the fatigue life of headed studs, and the relationship between the stress range and fatigue life could be explained as a line in double logarithm diagram.

Third, under the same stress loading ranges, the test result indicated that the studs under unidirectional loadings had a lower fatigue life than under reversal loadings. Since fracture always starts at the side of the stud which is subjected to bending tensile stress, it is not the full double amplitude under reversal stress, which is responsible for damage.

Finally, the flange thickness of the steel beam also affects the fatigue life of headed studs, the thicker the flange is, the lower fatigue life it would be.

Here, in Figure 5, the S-N curve is drawn to show primary features above. The design provision for shear studs from Eurocode 4 is drawn to have a comparison. The design equation in Eurocode 4 is:

$$\lg N + 8 \lg \Delta\tau = 21.93 \tag{1}$$

Although compared to Eurocode4 nowadays, these tests results got nearly 50 years ago are much lower, because of the poor weld technology that time, the fatigue features are clear.

Furthermore, the provision of AASHTO for resistance of shear studs is directly developed from

these results. The equation in AASHTO transformed to relationship of stress range and fatigue life is:

$$\Delta\tau = \frac{4}{\pi}(238 - 29.51\lg N) \tag{2}$$

The comparison of test results and the provision is shown in Figure 6.

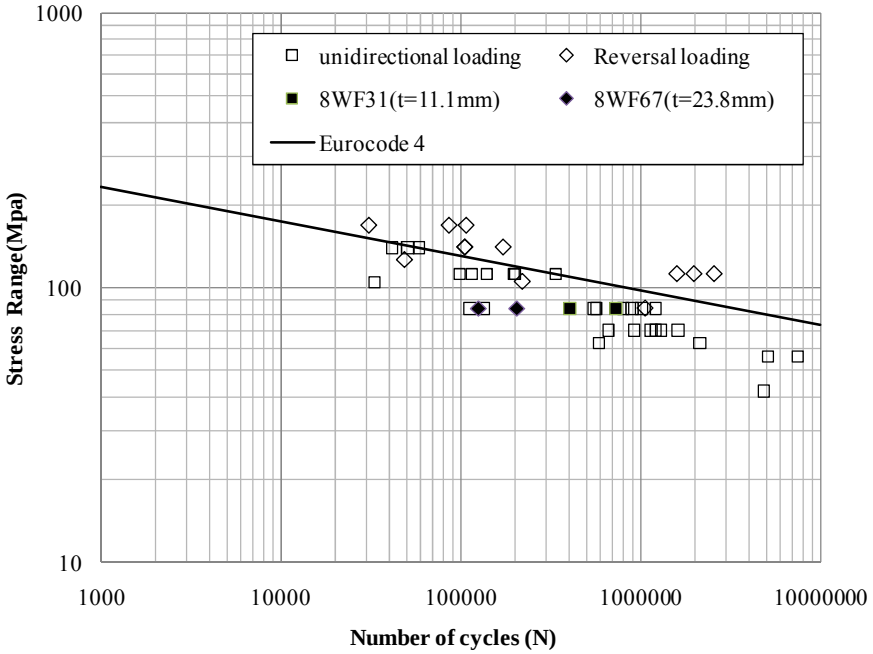


Figure 5 - S-N curves for stud test results by Slutter and Fisher

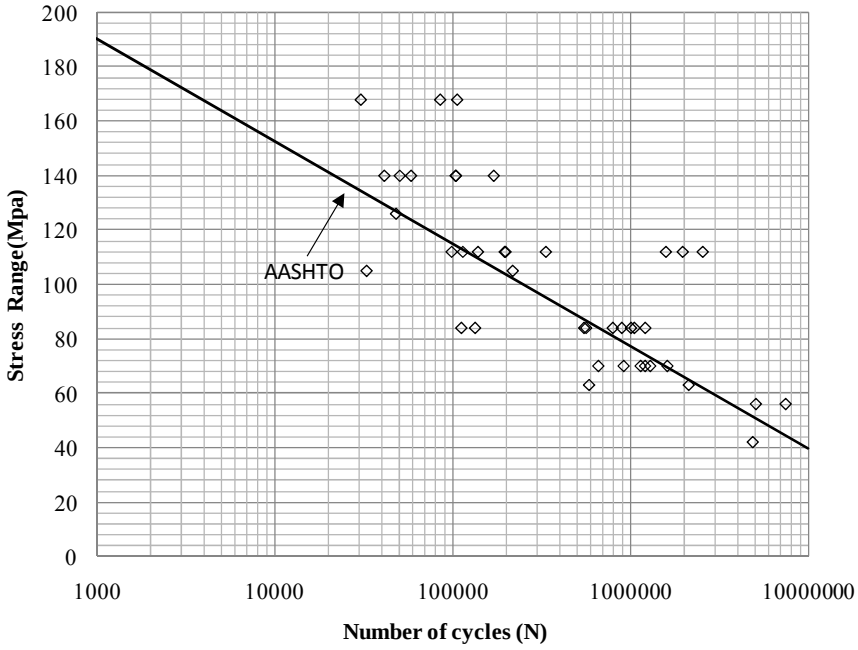


Figure 6 - Comparison of test results and the provision of AASHTO

2.1.2. Tests for channel connectors

Slutter and Fisher (1966) also examined the fatigue resistance on channel connector specimens by beam tests as shown in Figure 3.The channel connector specimens were tested following the same dispositions as the stud shear connectors. A total number of 12 specimens were tested and three of them were pilot tests. The typical fatigue failure of channel shear connectors was generally initiated in one of the transverse fillet weld and propagated through the weld. Since the fatigue always initiated in the connector nearest to the applied load, this channel carried more load and failed firstly as shown in Figure 7. And they proved that the fatigue strength of the single channel specimen was equivalent to the fatigue strength of specimens with two channel shear connectors. The orientation of the channel connector whether facing toward or away from the applied load had no significant influence on the fatigue life. The test details and results are listed up in Table 2. Similar to stud connector, the stress range is the main factor for fatigue life and the relationship of these two is described in equation below:

$$\Delta\tau = 1020 \cdot N^{-0.186} \tag{3}$$

The comparison of test results and the equation is shown in Figure 8.

Figure 7 - Failure models of channel connectors

Table 2 - Test results of channel connectors by Slutter and Fisher (1966)

	Test	f_c MPa	f_y MPa	f_u MPa	d mm	P_{max} kN	P_{min} kN	ΔP kN	$\Delta\tau$ MPa	N ($\times 10^3$)
1	d1D	41.7	342.1	423.8	152.4	80.3	-14.2	94.5	93	495.0
2	d2D	41.7	342.1	423.8	152.4	94.5	-13.2	107.7	106	9556.3
3	d3D	41.7	342.1	423.8	152.4	107.7	-13.2	120.9	119	1010.3
4	d2E	41.7	342.1	423.8	152.4	94.5	14.2	80.3	79	291.2
5	d3E	41.7	342.1	423.8	152.4	107.7	13.2	94.5	93	860.0
6	d4E	41.7	342.1	423.8	152.4	120.9	13.2	107.7	106	4000.0
7	d3F	41.7	342.1	423.8	152.4	107.7	40.6	67.1	66	460.0
8	d4F	41.7	342.1	423.8	152.4	120.9	40.6	80.3	79	1990.0
9	d5F	41.7	342.1	423.8	152.4	134.1	39.6	94.5	93	6550.0

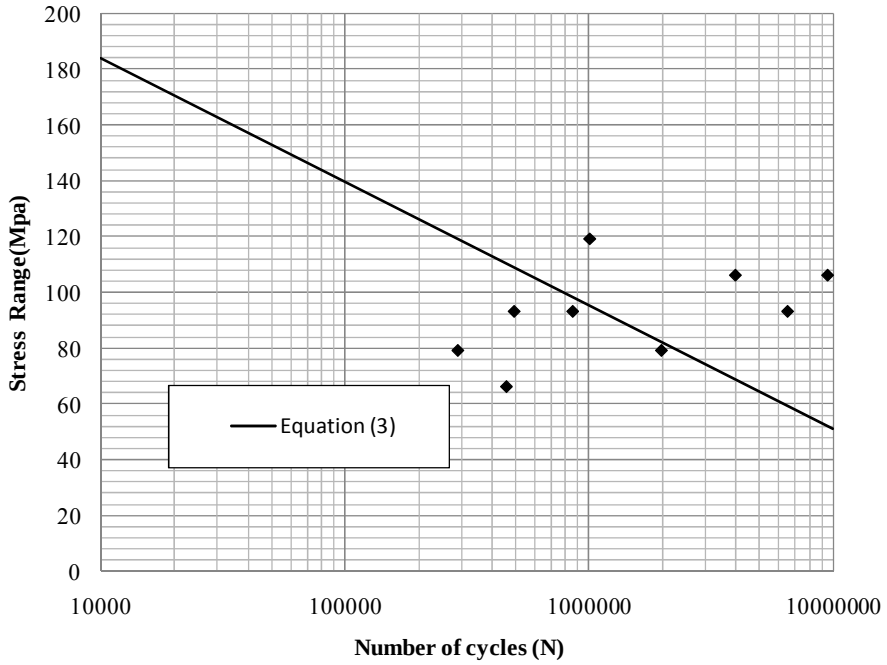


Figure 8 - Comparison of test results by Slutter and Fisher and the equation (3)

2.2. Tests by Mainstone and Menzies

Mainstone and Menzies (1967) performed fatigue tests on push-out specimens, using stud shear connectors, channel connectors and bar connectors. Only 23 tests results from stud shear connectors are collected and presented here. The specimens' geometry is in accordance with Figure 9, with two connectors on only one level. The specimen was not constrained from horizontal movement at its base. No details were given concerning the yielding and the ultimate strength of the rebars, the studs or the steel beams. Stud dimensions were 19 mm in diameter and 100 mm in height. It could be assumed for the headed studs usually used in Britain that the mean tensile strength of the stud material is about 600MPa. The test details and results are listed up in Table 3. And the relationship between stress range and the fatigue life is introduced in Figure 13.

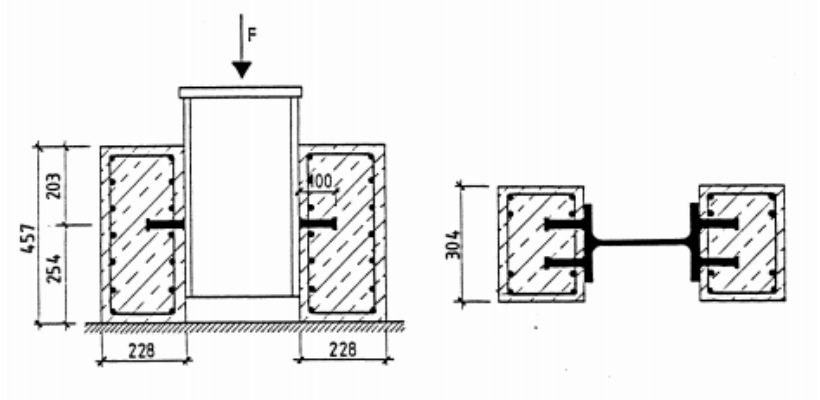


Figure 9 - Push-out-specimen used by Mainstone and Menzies (Roik and Hanswille 1990)

Table 3 - Test results of stud connectors by Mainstone and Menzies (1967)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	S1	27.4	-	600.0	19	49.9	4.8	45.1	159	76.0
2	S2	28.3	-	600.0	19	40.0	4.0	36.0	127	439.0
3	S7	29.5	-	600.0	19	40.0	4.0	36.0	127	1940.0
4	S9	31.2	-	600.0	19	55.0	5.4	49.6	175	42.0
5	S10	27.6	-	600.0	19	66.9	33.4	33.5	118	1700.0
6	S12	27.7	-	600.0	19	75.1	37.7	37.4	132	679.0
7	S13	22.3	-	600.0	19	20.1	-19.8	40.0	141	630.0
8	S15	28.5	-	600.0	19	20.1	-19.8	40.0	141	481.0
9	S17	34.8	-	600.0	19	17.6	-17.3	34.9	123	2200.0
10	S20	36.5	-	600.0	19	82.5	41.4	41.1	145	669.0
11	S21	36.5	-	600.0	19	34.9	-35.1	70.0	247	21.0
12	S23	30.0	-	600.0	19	85.1	63.8	21.3	75	657.0
13	S25	29.7	-	600.0	19	92.4	69.4	23.0	81	13.3
14	S27	30.6	-	600.0	19	87.6	43.9	43.7	154	9.0
15	S28	30.3	-	600.0	19	87.6	43.9	43.7	154	6.0
16	S31	26.8	-	600.0	19	85.1	42.5	42.6	150	8.6
17	S32	25.9	-	600.0	19	92.4	69.4	23.0	81	165.0
18	S33	25.5	-	600.0	19	89.9	67.4	22.4	79	106.0
19	S34	33.1	-	600.0	19	32.6	-28.3	60.9	215	16.5
20	P40	15.7	-	600.0	19	40.0	4.0	36.0	127	991.7
21	P41	15.8	-	600.0	19	40.0	4.0	36.0	127	650.9
22	P42	40.0	-	600.0	19	49.9	5.1	44.8	158	285.1
23	P43	40.0	-	600.0	19	49.9	5.1	44.8	158	143.0

2.3. Tests by Roderick and Ansourian

Roderick and Ansourian (1976) reported on 4 tests with one level load and three tests with multi-level load. Test specimens according to Figure 10 were examined. No details were given concerning the rebar. The studs presented 19 mm in diameter and 100 mm in height. The test details and results are listed up in Table 4. And the relationship between stress range and the fatigue life is introduced in Figure 13.

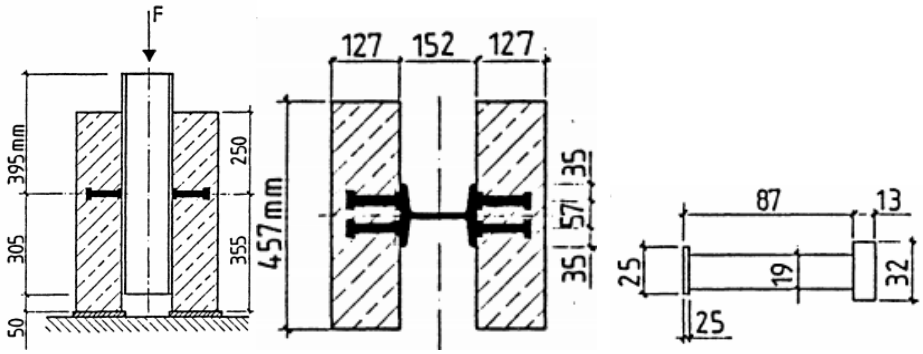


Figure 10 - Push-out-specimen used by Roderick and Ansourian (Roik and Hanswille 1990)

Table 4 - Test results by Roderick and Ansourian (1976)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	R1	26.4	274.0	466.0	19	44.2	4.5	39.7	140	616.0
2	R2	26.4	274.0	466.0	19	44.2	4.5	39.7	140	194.1
3	R3	26.4	274.0	466.0	19	44.2	4.5	39.7	140	190.5
4	R4	26.4	217.0	437.0	19	44.2	4.5	39.7	140	49.3

2.4. Tests by Hallam

Hallam (1976) reported on 13 tests using headed studs with 19 mm in diameter and 76 mm in height. Nothing is said about the rebar properties or the welding procedure. The push-out specimens were built with two studs on each side. And during the tests, the concrete slabs were prevented from separation by a steel rod, as shown in Figure 11.

When the first side failed, during the fatigue loading, the corresponding slab was removed, holes were drilled in the steel flange and a pre-cast concrete slab was bolted, replacing the original slab. The fatigue loading was then continued until the studs on the second side failed. The test details and results are listed up in Table 5. And the relationship between stress range and the fatigue life is introduced in Figure 13.

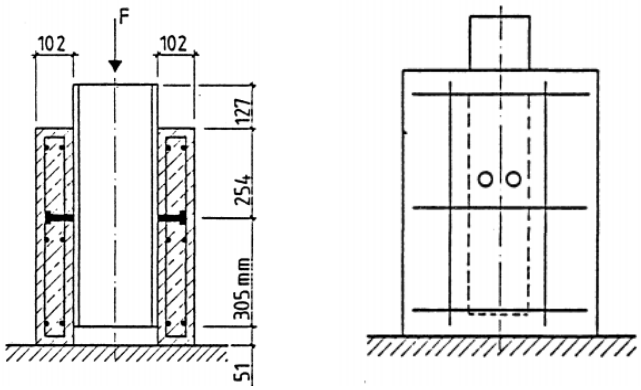


Figure 11 - Push-out-specimen used by Hallam (Roik and Hanswille 1990)

Table 5 - Test results by Hallam (1976)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	PS1	48.2	215.0	421.0	19	28.4	1.1	27.2	96	1303.0
2	PS2.1	46.5	215.0	421.0	19	28.4	2.3	26.1	92	824.0
3	PS2.2	46.5	215.0	421.0	19	28.4	2.3	26.1	92	845.0
4	PS3.1	46.5	215.0	421.0	19	28.4	2.3	26.1	92	652.0
5	PS3.2	46.5	215.0	421.0	19	28.4	2.3	26.1	92	652.0
6	PS4.1	28.1	215.0	421.0	19	49.9	2.3	47.6	168	52.8
7	PS4.2	28.1	215.0	421.0	19	49.9	2.3	47.6	168	52.8
8	PS5.1	28.1	215.0	421.0	19	49.9	2.3	47.6	168	58.6
9	PS5.2	28.1	215.0	421.0	19	49.9	2.3	47.6	168	67.9
10	PS6.1	26.5	215.0	421.0	19	23.2	2.3	21.0	74	3170.0
11	PS6.2	26.5	215.0	421.0	19	23.2	2.3	21.0	74	3554.0
12	PS7.1	26.5	215.0	421.0	19	23.2	2.3	21.0	74	5140.0
13	PS7.2	26.5	215.0	421.0	19	23.2	2.3	21.0	74	6096.0
14	PS8.1	26.2	215.0	421.0	19	23.0	1.1	21.8	77	21331.0
15	PS8.2	26.2	215.0	421.0	19	23.0	1.1	21.8	77	20965.0
16	PS9.1	26.2	215.0	421.0	19	23.0	1.1	21.8	77	24305.0
17	PS9.2	26.2	215.0	421.0	19	23.0	1.1	21.8	77	35000.0
18	PS10.1	24.2	215.0	421.0	19	44.2	2.3	42.0	148	61.7
19	PS10.2	24.2	215.0	421.0	19	44.2	2.3	42.0	148	75.5
20	PS11.1	24.2	215.0	421.0	19	44.2	2.3	42.0	148	110.0
21	PS11.2	24.2	215.0	421.0	19	44.2	2.3	42.0	148	110.0
22	PS12.1	17.8	215.0	421.0	19	33.7	2.3	31.5	111	148.7
23	PS12.2	17.8	215.0	421.0	19	33.7	2.3	31.5	111	174.8
24	PS13.1	17.8	215.0	421.0	19	33.7	2.3	31.5	111	182.6
25	PS13.2	17.8	215.0	421.0	19	33.7	2.3	31.5	111	182.6

2.5. Tests by Roik and Holtkamp

Roik and Holtkamp (1976) reported on push-out-tests under pure shear stress. The tested specimens are represented in Figure 12. The diameter of rebar was 7.5 mm. 4 transversal and 4 longitudinal rebars were disposed on the concrete slab. The test details and results are listed up in Table 6. And the relationship between stress range and the fatigue life is introduced in Figure 13.

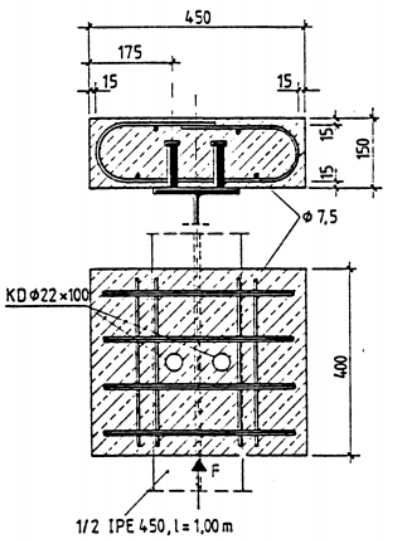


Figure 12 - Push-out-specimen used by Roik and Holtkamp (Roik and Hanswille 1990)

Table 6 - Test results by Roik and Holtkamp (1976)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	3	22.5	375.0	492.0	22	52.5	5.3	47.1	124	4030.0
2	4	22.5	375.0	492.0	22	69.9	9.9	60.1	158	640.0
3	5	22.5	375.0	492.0	22	71.8	10.3	61.6	162	160.0
4	6	22.5	375.0	492.0	22	71.8	9.9	62.0	163	220.0
5	7	22.5	375.0	492.0	22	71.5	10.3	61.2	161	480.0

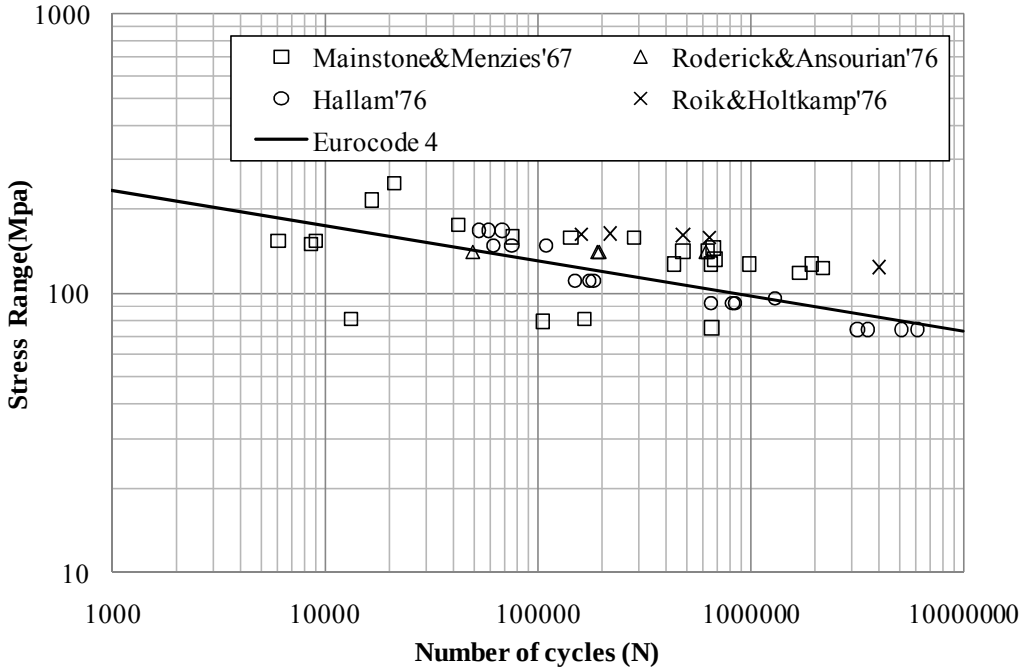
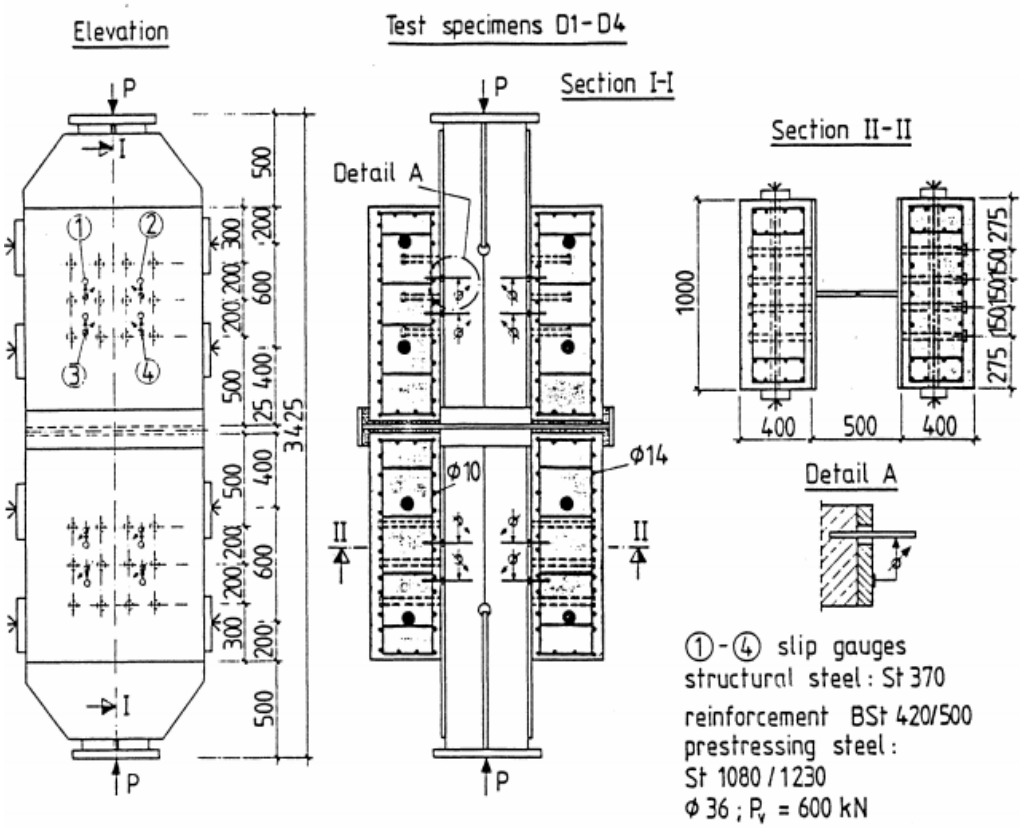


Figure 13 - S-N relationships of tests by authors in 1960s to 1970s

2.6. Tests by Roik and Hanswille

Roik and Hanswille (1989) reported on tests with concrete flanges under compressive stress and with cracked concrete flanges in tension. The specimens are shown in Figure 14. The setups were a little different from normal push-out tests. For test specimens D1-D2, actually there were two specimens connected together at the end, one in the usual direction, the other opposite; the compressive load was applied at the top and transmitted from one beam to another, so more studs could be tested at one time. And two prestressing steel bars passed throw each concrete beam, the prestressing load was 600 kN, For test specimens Z1-Z3, one specimen were used and the tensile load was applied at the top with end fixed. For Z1, two prestressing steel bars passed throw each concrete beam, the prestressing load was 600 kN too. So the compression tests D1, D2 and the tension test Z1 were prestressed in transverse direction; the tests Z2 and Z3 had no transverse prestressing. In Table 7 and Figure 15 the test details and results are listed up.
In figure 15, it is clear that the endurance increases remarkably due to transverse compression. But because of the scarce test numbers, the results are not sufficient for a safe statement.



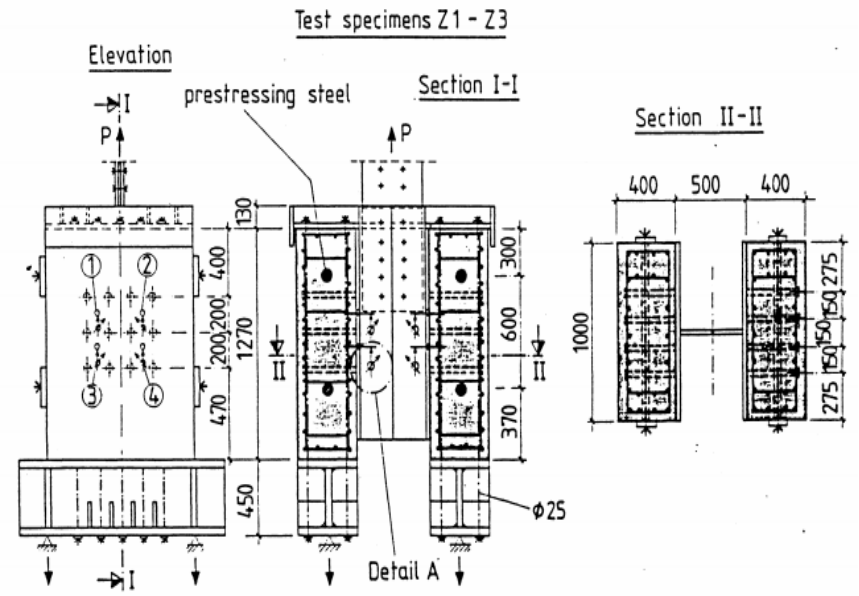


Figure 14 - Push-out-specimen used by Roik and Hanswille (Roik and Hanswille 1990)

Table 7 - Test results by Roik and Hanswille (1989)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	D1	40.8	373.0	485.0	22	62.3	8.0	54.4	143	2875.0
2	D2	40.8	373.0	485.0	22	62.3	8.0	54.4	143	2550.0
3	Z1	51.3	373.0	485.0	22	70.7	8.4	62.3	164	2940.0
4	Z2	32.5	373.0	485.0	22	70.7	8.4	62.3	164	170.0
5	Z3	31.5	373.0	485.0	22	70.7	8.4	62.3	164	1335.0

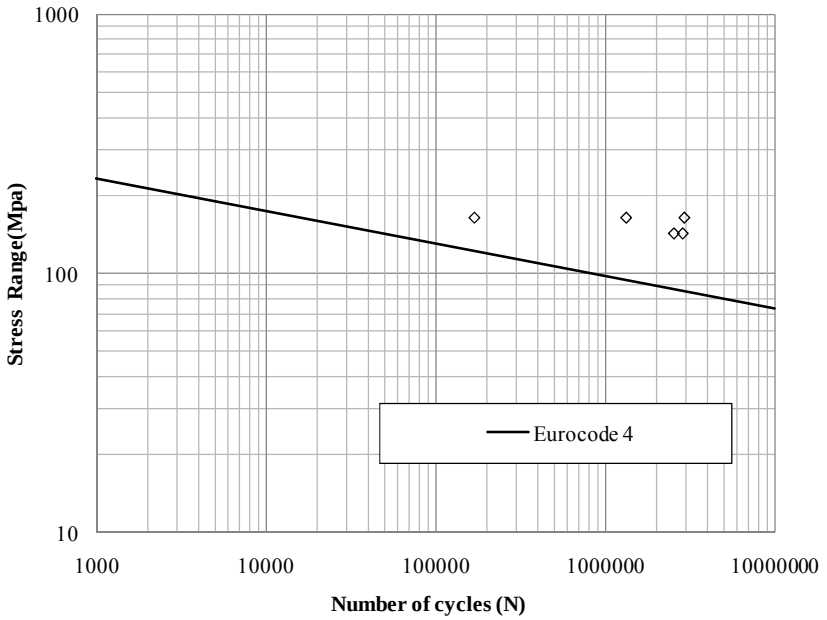


Figure 15 - Test results by Roik and Hanswille (1989)

2.7. Tests by Naithani and Gupta

Naithani and Gupta (1988) reported on 12 specimens with stud shear connectors tested with a new kind of test set-up as shown in Figure 16. In this kind of test set-up, the shear transfer at the interface of steel and concrete may be unaccompanied by moment, and it was unaffected by 'group effect'. A group of 12 specimens was subjected to uniform conditions of fatigue loading. These were tested for varying stress ranges, keeping the concrete strength approximately the same. In Table 8 the test details and results are listed up.

Based on the tests, the following relationship had resulted between the life and stress range for an 18 mm diameter stud connectors by authors, as:

$$\log N = 7.595 - 0.02827\Delta\tau \tag{4}$$

The tests results dotted in Figure 17 are compared to AASHTO and the Slutter and Fisher's results, obviously, the former results are lower to AASHTO and the latter, because of the difference between test methods and the disappearance of 'group effect'. If equation (4) applied in design, the values of stress range obtained by this equation would result in a lower value and may lead to be a much safer design.

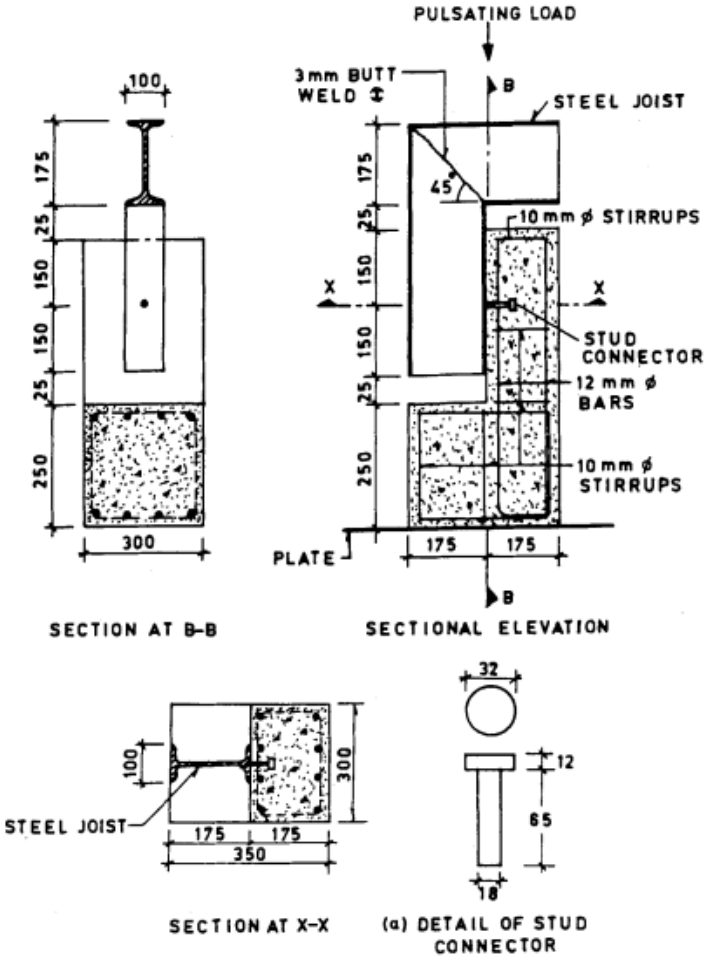


Figure 16 - New Standard test specimen used by Naithani and Gupta (1988)

Table 8 - Test results by Naithani and Gupta (1988)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	F1	26.8	295.0	450.0	18	49.2	10.0	39.2	154.2	1.0
2	F2	25.0	295.0	450.0	18	49.2	10.0	39.2	154.2	3.2
3	F3	31.5	295.0	450.0	18	39.4	10.0	29.4	115.65	23.0
4	F4	30.5	295.0	450.0	18	39.4	10.0	29.4	115.65	21.0
5	F5	20.0	295.0	450.0	18	34.5	10.0	24.5	96.38	68.0
6	F6	20.8	295.0	450.0	18	34.5	10.0	24.5	96.38	78.0
7	F7	25.0	295.0	450.0	18	29.6	10.0	19.6	77.1	266.0
8	F8	26.8	295.0	450.0	18	29.6	10.0	19.6	77.1	48.0
9	F9	21.7	295.0	450.0	18	24.7	10.0	14.7	57.83	1150.0
10	F10	20.6	295.0	450.0	18	24.7	10.0	14.7	57.83	685.0
11	F11	28.0	295.0	450.0	18	22.3	10.0	12.3	48.2	2000.0*
12	F11'	28.0	295.0	450.0	18	24.7	10.0	14.7	57.83	1247.5
13	F12	26.7	295.0	450.0	18	19.8	10.0	9.8	38.55	2512.0*

* No failure

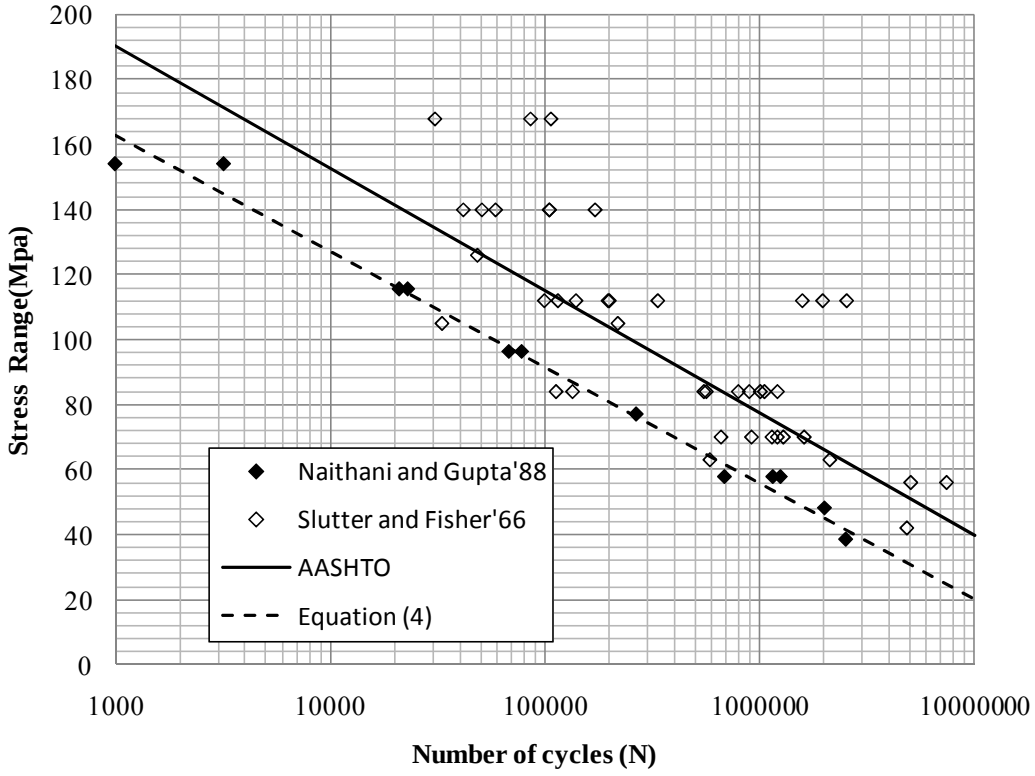


Figure 17 - Test results by Naithani and Gupta (1988)

2.8. Tests by Faust and Leffer

Faust and Leffer (2000) reported push-out tests performed with headed studs embedded in lightweight aggregate concrete (LWAC). The specimens were tested under cyclic loading and the test set-up is shown in Figure 18. Four headed studs with 22 mm diameter and 125 mm height were welded on the flanges of the steel section. Two steel bars were attached at the end in order to take over tensile forces which tend to separate the slab from the steel beam. No details were given concerning the rebar. The test details and results are listed up in Table 9 and Figure 17.Compared to design provision in Eurocde 4 for headed studs in normal aggregate concrete, the fatigue strength curve in somewhat more inclined in case of LWAC. And so far the results conducted that the Eurocode 4 is conservative for LWAC with closed structure.

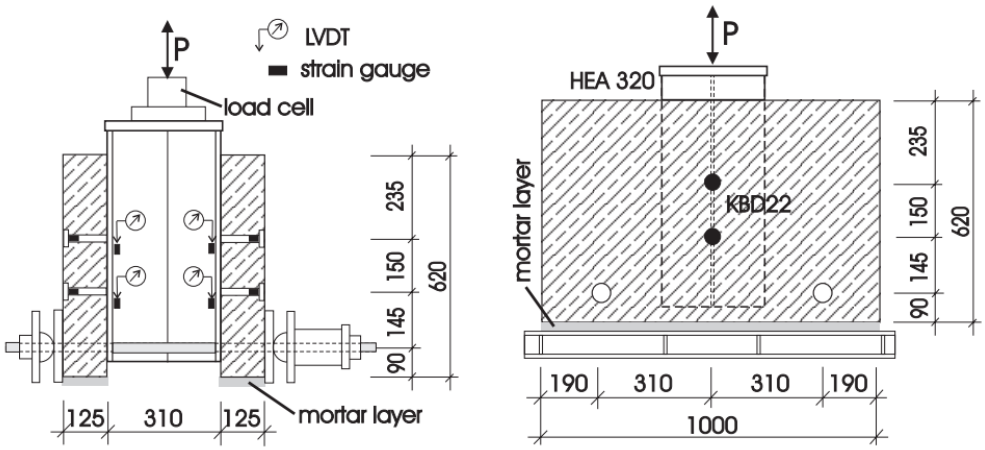


Figure 18 - Push-out-specimen used by Faust and Leffer (2000)

Table 9 - Test results by Faust and Leffer (2000)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	1	48.00	-	-	22	69	26	42	110.9	566.0
2	2	48.00	-	-	22	69	26	42	110.9	522.6
3	3	48.00	-	-	22	69	25	44	116.0	720.0
4	4	48.00	-	-	22	69	15	54	141.8	83.7
5	5	48.00	-	-	22	69	14	55	144.4	103.6
6	6	48.00	-	-	22	69	13	56	147.0	96.5
7	7	48.00	-	-	22	69	5	64	167.6	60.4
8	8	48.00	-	-	22	69	27	41	108.3	550.0
9	9	48.00	-	-	22	69	29	39	103.2	907.0
10	10	48.00	-	-	22	69	29	39	103.2	913.0
11	11	48.00	-	-	22	69	7	62	162.5	39.1

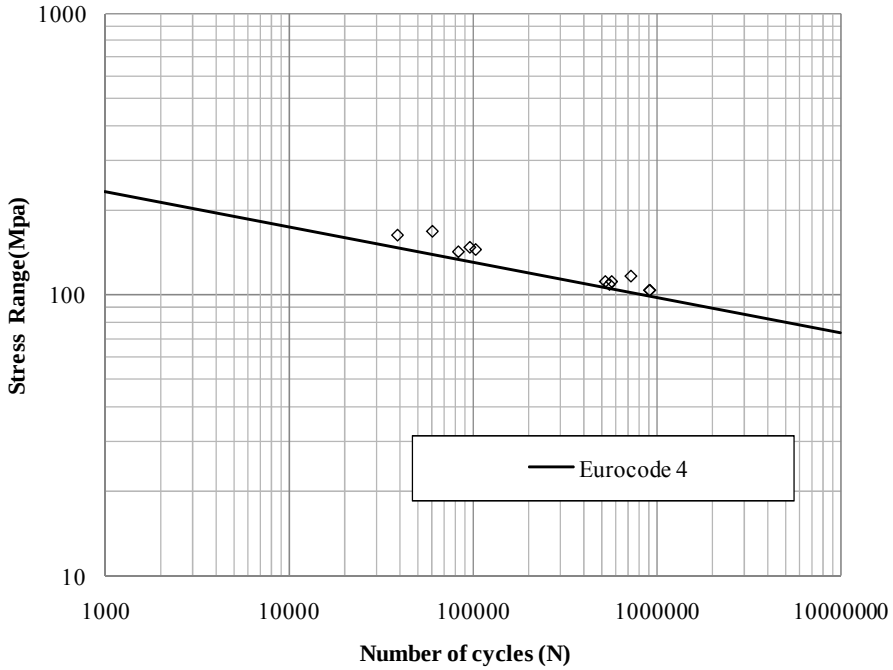


Figure 19 - Test results by Faust and Leffer (2000)

2.9. Tests by Shim and Lee

The fatigue endurance of the shear connection applied in composite bridges with precast decks has been reported by Shim and Lee (2001). The standard push-out specimens were built according to Figure 20. The studs had 19 mm in diameter and 150 mm in height. The specimens were performed with a 20 mm bedding layer to estimate the fatigue endurance in a precast deck. Since usually the thickness of the bedding layer in a real structure is 20 mm. No information about rebar was given. The test details and results are listed up in Table 10.

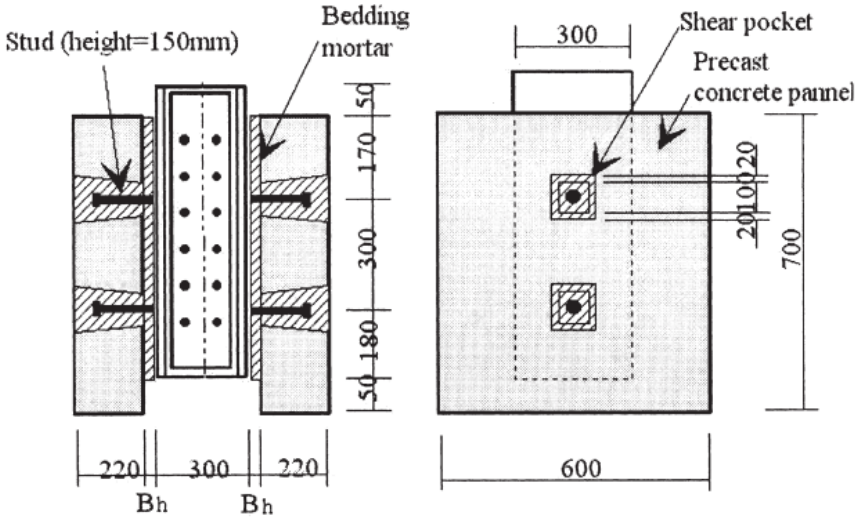


Figure 20 - Push-out-specimen used by Shim and Lee (2001)

Table 10 -Test results by Shim and Lee (2001)

	Test	f_c MPa	f_y MPa	f_u MPa	d mm	P_{max} kN	P_{min} kN	ΔP kN	$\Delta\tau$ MPa	N ($\times 10^3$)
1	F150A	54.88	-	450.0	19	36.75	1.25	35.50	125.2	75.0
2	F170A	54.88	-	450.0	19	41.66	1.25	40.40	142.5	68.0
3	F130B	61.09	-	450.0	19	31.85	1.25	30.59	107.9	380.0
4	F150B	61.09	-	450.0	19	36.75	1.25	35.50	125.2	210.0
5	F180B	61.09	-	450.0	19	44.10	1.25	42.84	151.1	63.0
6	F130C	71.38	-	450.0	19	31.85	1.25	30.59	107.9	410.0
7	F150C	71.38	-	450.0	19	36.75	1.25	35.50	125.2	490.0
8	F180C	71.38	-	450.0	19	44.10	1.25	42.84	151.1	49.0
9	F180C	71.38	-	450.0	19	44.10	1.25	42.84	151.1	60.0

There was no massage about the failure model of this test, but based on the test results, the empirical equation of the fatigue endurance is suggested as:

$$\log N = 7.8869 - 0.021\Delta\tau \tag{5}$$

In Figure 21, the comparison of this equation and AASHTO is shown. The results seem to be much higher compared to AASHTO. Furthermore, further experiments for the effect of bedding thickness are necessary on the fatigue endurance of shear connections in a precast deck.

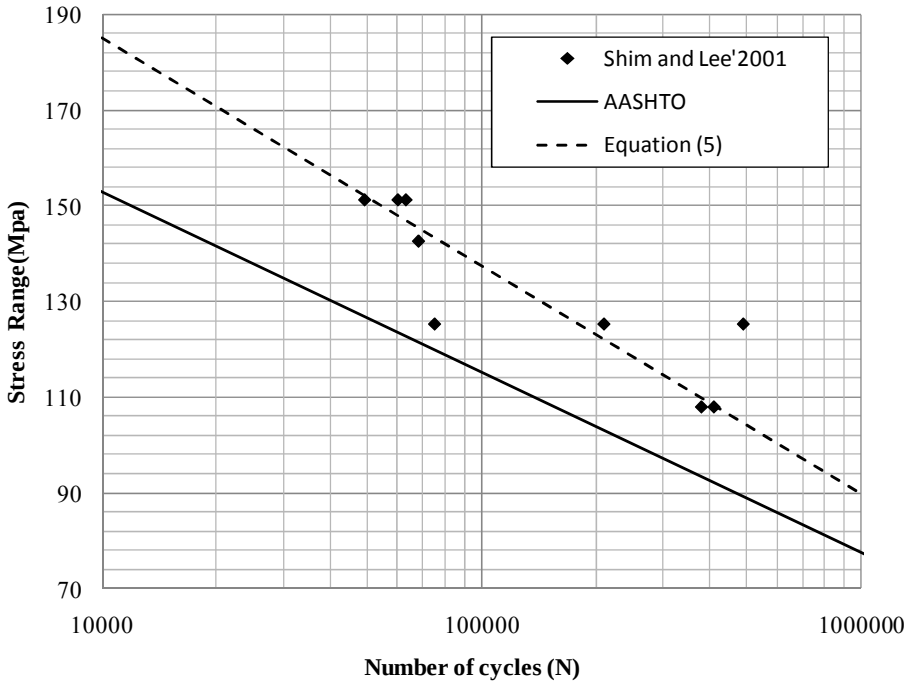


Figure 21 - Test results by Shim and Lee (2001)

2.10. Tests by Badie and Tadros

Badie and Tadros (2002) reported on fatigue tests performed with large shear studs in steel

bridge girders. The diameter of the steel studs was 31.8 mm. these kinds of studs had about twice the static ultimate strength than the 22.2 mm studs, so fewer studs were required along the height of steel girder. L-shaped specimens with different kinds of rebar were examined as shown in Figure 22. The height of studs was 140 mm. During the tests, fatigue loading was applied until two million cycles were reached or failure occurred; whichever occurred first. If the test specimen endured the two million cycles without failure, a maximum monotonic load would be applied. During this monotonic loading, slip and load were recorded. The test details and results are listed up in Table 11.

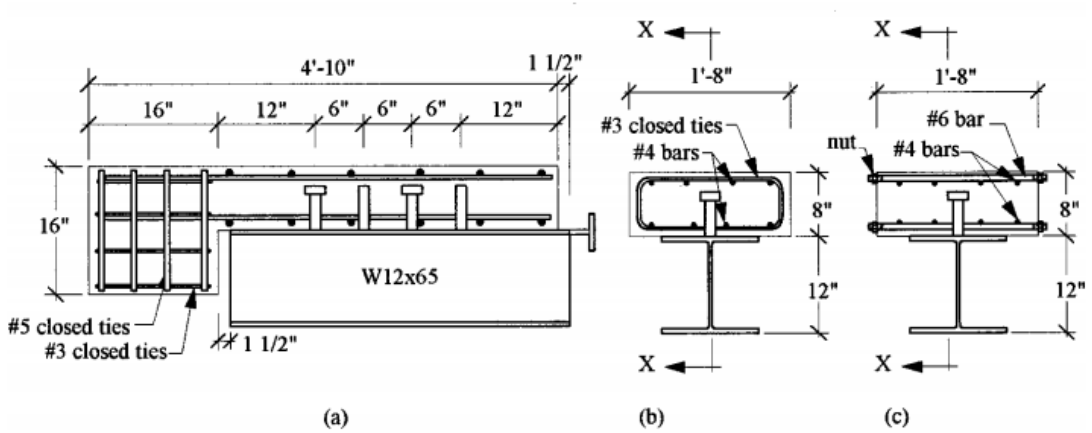


Figure 22 - L-shaped specimen used by Badie and Tadros (2002)

Table 11 - Test results by Badie and Tadros (2002)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	LS-5-25A	48.00	340.0	440.0	31.8	164	27	137	172.4	49.0
2	LS-5-25B	48.00	340.0	440.0	31.8	164	27	137	172.4	50.0
3	LS-5-23A	48.00	340.0	440.0	31.8	153	27	126	158.6	74.0
4	LS-5-21	48.00	340.0	440.0	31.8	142	27	115	144.8	94.0
5	LS-5-20	48.00	340.0	440.0	31.8	137	27	110	137.9	554.0
6	LS-5-19	48.00	340.0	440.0	31.8	131	27	104	131.0	567.0
7	LS5-18A	48.00	340.0	440.0	31.8	126	27	99	124.1	166.0
8	LS5-18B	48.00	340.0	440.0	31.8	126	27	99	124.1	2533.0
9	LS-5-17	48.00	340.0	440.0	31.8	120	27	93	117.2	1636.0
10	LS-5-16	48.00	340.0	440.0	31.8	115	27	88	110.3	1372.0
11	LS-5-15	48.00	340.0	440.0	31.8	110	27	82	103.4	2000.0*
12	LS-5-14	48.00	340.0	440.0	31.8	104	27	77	96.5	2594.0*
13	LS-5-10A	48.00	340.0	440.0	31.8	82	27	55	68.9	4680.0*
14	LS-5-10B	48.00	340.0	440.0	31.8	82	27	55	68.9	6708.0*
15	SS-5-25	48.00	290.0	340.0	22.2	80	13	67	172.4	27.0

16	SS-5-23	48.00	290.0	340.0	22.2	75	13	61	158.6	60.0
17	SS-5-21	48.00	290.0	340.0	22.2	69	13	56	144.8	285.0
18	SS-5-20	48.00	290.0	340.0	22.2	67	13	53	137.9	189.0
19	SS-5-19	48.00	290.0	340.0	22.2	64	13	51	131.0	157.0
20	SS-5-18	48.00	290.0	340.0	22.2	61	13	48	124.1	935.0
21	SS-5-17	48.00	290.0	340.0	22.2	59	13	45	117.2	400.0
22	SS-5-16	48.00	290.0	340.0	22.2	56	13	43	110.3	245.2
23	SS-5-15	48.00	290.0	340.0	22.2	53	13	40	103.4	600.0
24	SS-5-14	48.00	290.0	340.0	22.2	51	13	37	96.5	2000.0*
25	SS-5-10	48.00	290.0	340.0	22.2	40	13	27	68.9	2500.0*

* No failure

2.11. Tests by Seracino and Oehlers

A new push-pull specimen was developed by Seracino and Oehlers (2003) to provide a better understanding of bi-directional cyclic fatigue behavior of stud shear connectors. The pull-push test setup is shown in Figure 23. The specimen was restrained by four supports located at each corner of the concrete block. No internal reinforcement was placed near the connectors. Three stud shear connectors with 75 mm height and 12.7 mm diameter were welded onto a steel plate (flange). The studs presented 300 mm spacing and were embedded in concrete. The test details and results are listed up in Table 12 and Figure 24. The tests confirmed that for a given range of load, the fatigue life of connectors subjected to bi-directional cyclic loading is normally longer than those subjected to unidirectional loads only, with the exception of two bi-directional tests, F14-P2 and F15-P2 where $P_{max}/\Delta P$=0.85. Similarly, it proved the fatigue characteristics gotten from test results of Slutter and Fisher (1966) and Mainstone and Menzies (1967).

It was also shown in this test that the rate of increase in slip per cycle is constant over most of the fatigue life with a rapid increase near the end so that, if monitored, can be used to provide adequate warning of failure.

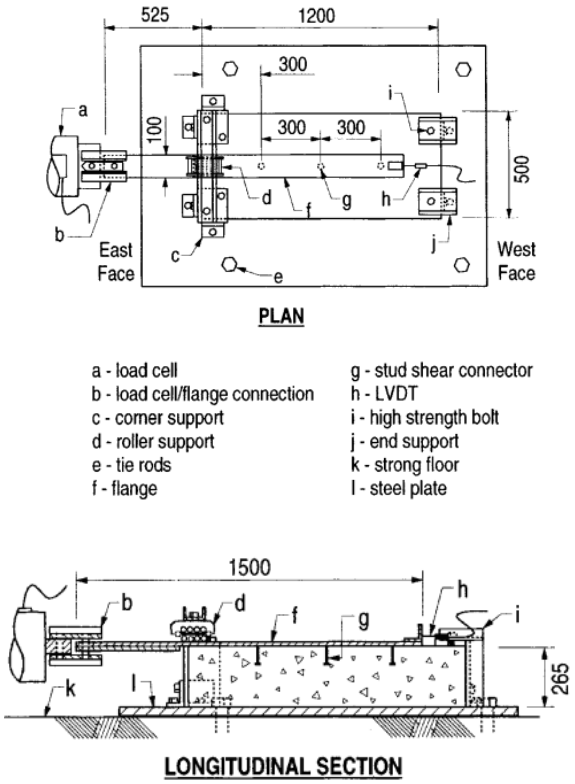


Figure 23 - Push-out-specimen used by Seracino and Oehlers (2003)

Table 12 - Test results by Seracino and Oehlers (2003)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	F1-P1	35.20	-	433.0	12.7	6.7	-6.7	13.3	105.3	1081.9
2	F2-P1	35.20	-	433.0	12.7	8.3	-8.3	16.7	131.6	789.5
3	F3-P1	35.20	-	433.0	12.7	10.0	-10.0	20.0	158.0	75.0
4	F4-P1	35.20	-	433.0	12.7	11.7	-11.7	23.3	184.3	25.8
5	F5-P1	35.20	-	433.0	12.7	23.3	3.3	20.0	158.0	29.6
6	F6-P1	35.20	-	433.0	12.7	20.0	3.3	16.7	131.6	144.4
7	F7-P2	40.20	-	433.0	12.7	16.7	3.3	13.3	105.3	831.7
8	F8-P2	40.20	-	433.0	12.7	8.3	-8.3	16.7	131.6	279.6
9	F9-P2	40.20	-	433.0	12.7	8.3	-8.3	16.7	131.6	2296.2
10	F10-P2	40.20	-	433.0	12.7	20.0	3.3	16.7	131.6	158.9
11	F11-P2	40.20	-	433.0	12.7	12.5	-4.2	16.7	131.6	930.8
12	F12-P2	40.20	-	433.0	12.7	12.5	-4.2	16.7	131.6	263.1
13	F13-P2	40.20	-	433.0	12.7	8.3	-8.3	16.7	131.6	793.4
14	F14-P2	40.20	-	433.0	12.7	14.2	-2.5	16.7	131.6	69.7
15	F15-P2	40.20	-	433.0	12.7	14.2	-2.5	16.7	131.6	109.0

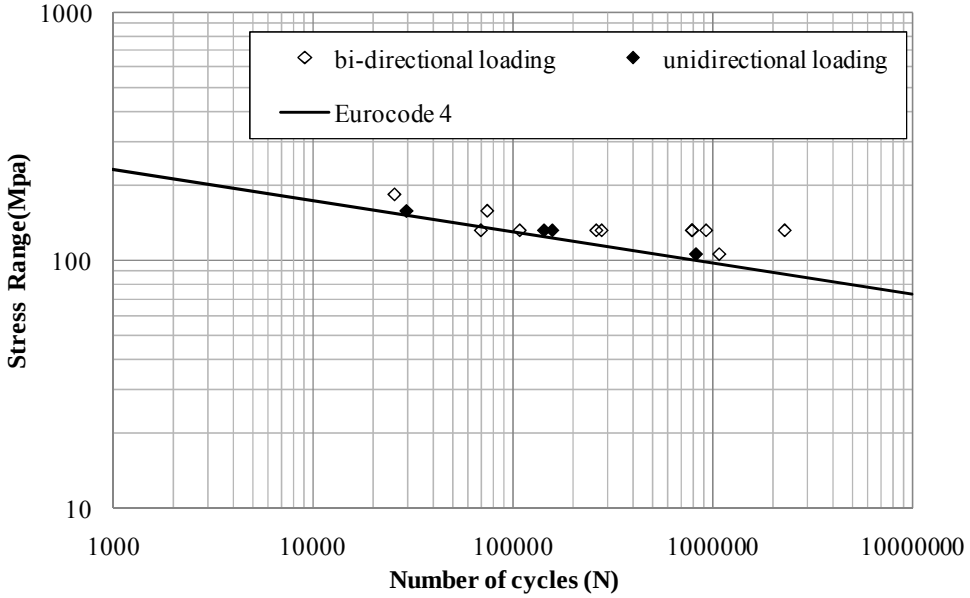


Figure 24 - Test results by Seracino and Oehlers (2003)

2.12. Tests by Lee and Shim

Lee and Shim (2005) reported on fatigue tests of large stud shear connectors, namely the diameters of 25 mm, 27 mm and 30 mm. Standard push-out specimens were tested as shown in Figure 25. The rebar had 16 mm diameter. The studs presented 155 mm height. The test details and results are listed up in Table 13.

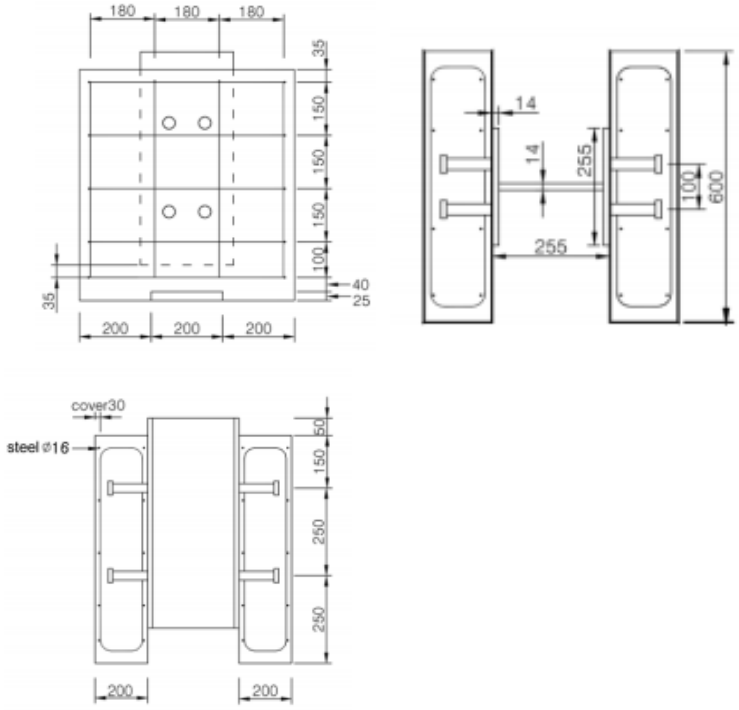


Figure 25 - Push-out-specimen used by Lee and Shim (2005)

Table 13 - Test results by Lee and Shim (2005)

	Test	f_c MPa	f_y MPa	f_u MPa	d mm	P_{max} kN	P_{min} kN	ΔP kN	$\Delta\tau$ MPa	N ($\times 10^3$)
1	FT25-A1	30.0	353.0	426.0	25	49.1	0.0	49.1	100	2134.0
2	FT25-A2	30.0	353.0	426.0	25	73.6	0.0	73.6	150	44.8
3	FT25-A3	30.0	353.0	426.0	25	83.4	0.0	83.4	170	60.0
4	FT25-B1	40.0	353.0	426.0	25	63.8	0.0	63.8	130	387.2
5	FT25-B2	40.0	353.0	426.0	25	73.6	0.0	73.6	150	61.1
6	FT25-B3	40.0	353.0	426.0	25	87.0	0.0	87.0	177.3	5.3
7	FT27-A1	30.0	353.0	426.0	27	73.5	0.0	73.5	128.4	142.6
8	FT27-A2	30.0	353.0	426.0	27	85.9	0.0	85.9	150	22.5
9	FT27-A3	30.0	353.0	426.0	27	97.3	0.0	97.3	170	13.8
10	FT30-A1	30.0	353.0	426.0	30	91.9	0.0	91.9	130	75.5
11	FT30-A2	30.0	353.0	426.0	30	106.0	0.0	106.0	150	10.4
12	FT30-A3	30.0	353.0	426.0	30	110.3	0.0	110.3	156.1	19.3

The test results including the test results by Badie and Tadros (2002) are plotted in Figure 26. It was proved that fatigue endurance obtained from the tests was slightly lower than the current S-N curve in eurocode 4.

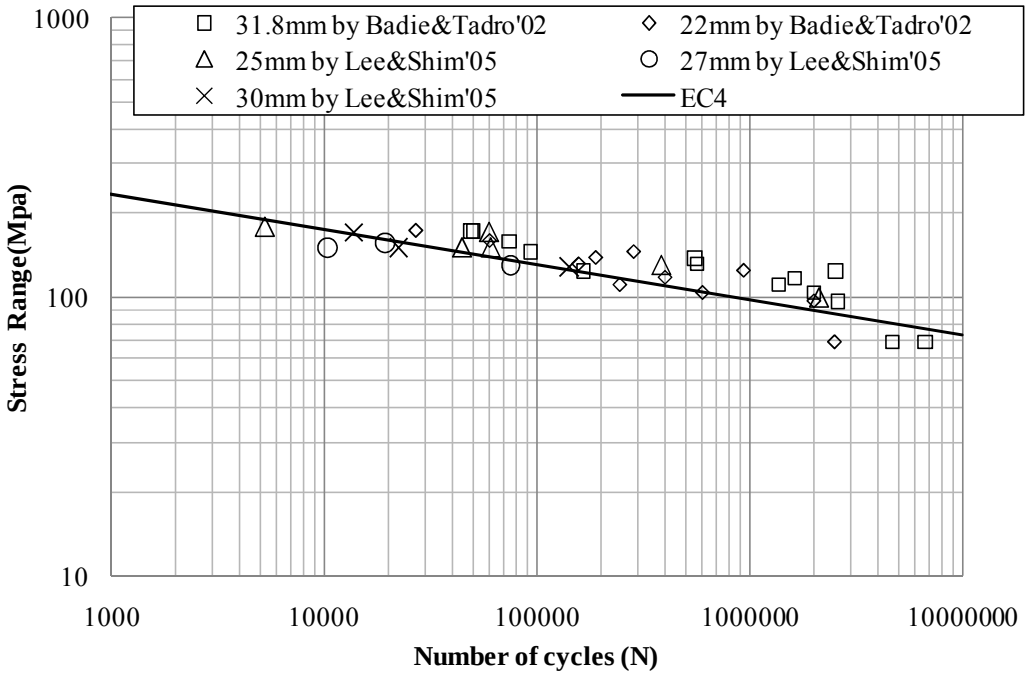


Figure 26 - Relation between stress range and fatigue life for large studs

2.13. Tests by Ahn and Kim

Ahn and Kim (2007) reported on fatigue experiments of studs welded on steel plate for a new bridge deck system. Nine specimens were fabricated according to the standard push-out specimen for endurance testing in EN1994-1-1 as shown in Figure 27. The concrete slabs were connected to 9 mm steel plates by means of four welded stud shear connectors with 16 mm diameter and 125 mm height. The rebars had 16 mm in diameter. The test details and results are listed up in Table 14 and Figure 28. The comparison between these tests results and the Eurocode 4 shows that the fatigue endurance of the specimens satisfies the design value of code.

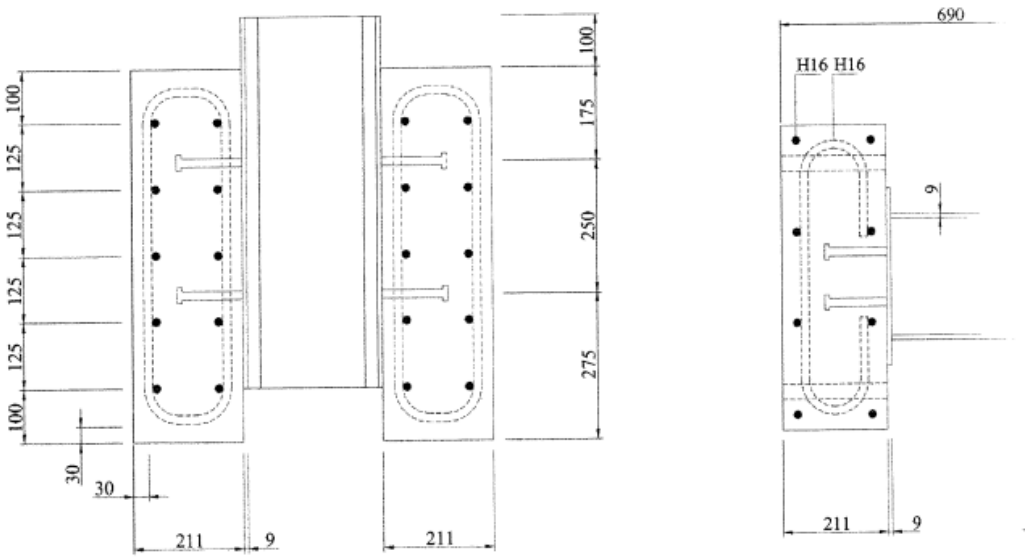


Figure 27 - Push-out-specimen used by Ahn and Kim (2007)

Table 14 - Test results by Ahn and Kim (2007)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	ST-F-A1	30.0	351.0	422.0	16	20.1	0.0	20.1	100	2120.0
2	ST-F-A2	30.0	351.0	422.0	16	20.1	0.0	20.1	100	2535.5
3	ST-F-A3	30.0	351.0	422.0	16	20.1	0.0	20.1	100	2828.6
4	ST-F-B1	30.0	351.0	422.0	16	26.1	0.0	26.1	130	656.9
5	ST-F-B2	30.0	351.0	422.0	16	26.1	0.0	26.1	130	735.7
6	ST-F-B3	30.0	351.0	422.0	16	26.1	0.0	26.1	130	1300.8
7	ST-F-C1	30.0	351.0	422.0	16	30.2	0.0	30.2	150	231.6
8	ST-F-C2	30.0	351.0	422.0	16	30.2	0.0	30.2	150	274.4
9	ST-F-C3	30.0	351.0	422.0	16	30.2	0.0	30.2	150	161.4

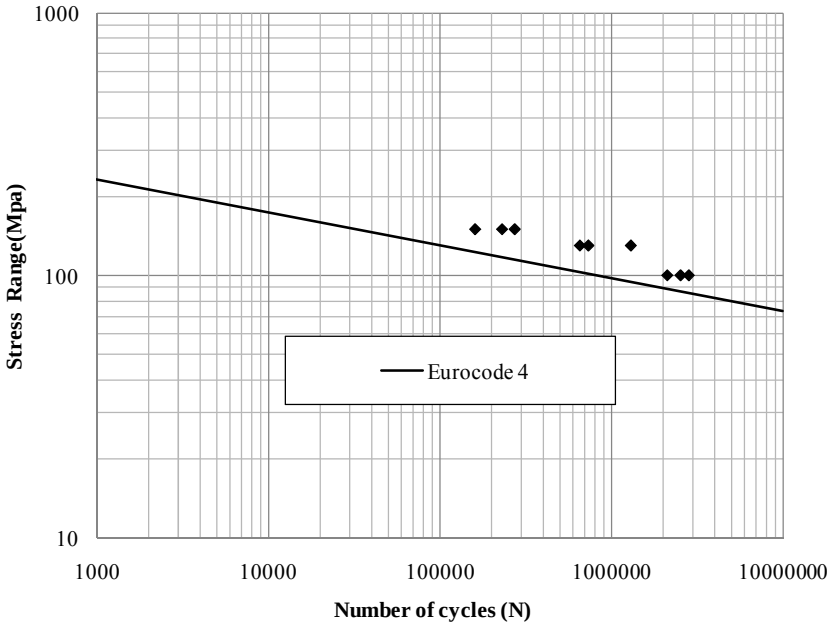


Figure 28 – Test results by Ahn and Kim (2007)

2.14. Tests by Kwon and Engelhardt

Kwon and Engelhardt (2010) reported on fatigue tests of post-installed shear connectors, which were applied to develop composite action in existing non-composite floor systems. Three types of post-installed shear connectors with 22 mm diameter were tested. The shear connectors' configuration is shown in Figure 29. A kind of direct-shear test setup for single shear connector tests and the reinforcing bar layout of single shear connector test specimens were shown in Figure 30. All post-installed shear connectors had a 127 mm embedment depth into the 178 mm thick concrete block. In Table 15 the test details and results are listed up. The empirical equation established by authors is:

$$\Delta\tau = 536 - 60.01 \log N \tag{6}$$

The comparison for the equation and AASHTO is shown in Figure 31. It proved that the post-installed shear connectors showed significantly higher fatigue strength than conventional welded shear studs. The superior fatigue strength of these post-installed shear connectors enables the strengthening of existing bridges by using significantly fewer shear connectors than would be possible with conventional welded shear studs.

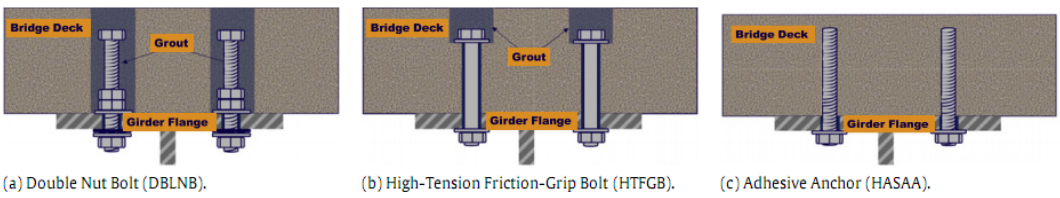


Figure 29 - Post-installed shear connectors (Kwon and Engelhardt 2010)

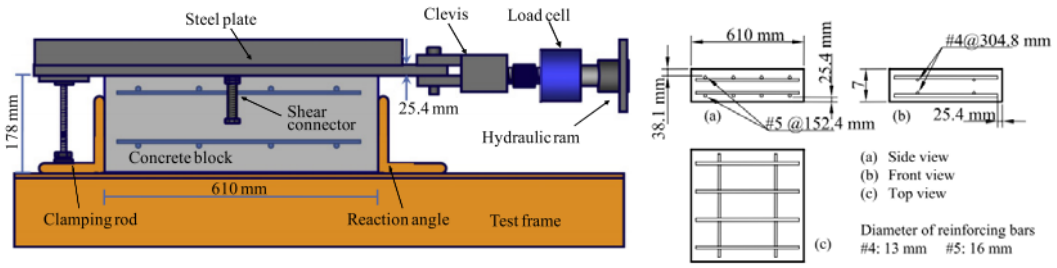


Figure 30 - Direct-shear test setup and reinforcing bar layout (Kwon and Engelhardt 2010)

Table 15 - Test results by Kwon and Engelhardt (2010)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta\tau$	N
		MPa	MPa	MPa	mm	kN	kN	kN	MPa	($\times 10^3$)
1	HASAA-15HF	44.3	-	724.0	22	43.2	4.0	39.2	103	10000.0*
2	HASAA-20HF	26.5	-	724.0	22	56.5	4.0	52.5	138	4362.9
3	HASAA-25HF	26.5	-	724.0	22	69.4	4.0	65.4	172	114.4
4	HASAA-25HF1	26.5	-	724.0	22	69.4	4.0	65.4	172	164.0
5	HASAA-30HF	26.5	-	724.0	22	82.7	4.0	78.7	207	341.6
6	HASAA-30HF1	23.5	-	628.0	22	82.7	4.0	78.7	207	58.2
7	HASAA-35HF	23.5	-	628.0	22	95.6	4.0	91.6	241	210.3
8	HASAA-40HF	23.5	-	628.0	22	108.9	4.0	104.9	276	56.1
9	DBLNB-45HF	41.6	-	724.0	22	121.8	4.0	117.8	310	5112.0*
10	HTFGB-35HF1	43.0	-	604.0	22	95.6	4.0	91.6	241	5002.7*
11	HASAA-30HF2	23.8	-	725.0	22	82.7	4.0	78.7	207	9.8
12	HASAA-30HF3	23.8	-	725.0	22	82.7	4.0	78.7	207	21.5
13	HASAA-35HF1	23.8	-	725.0	22	95.6	4.0	91.6	241	101.8

* No failure

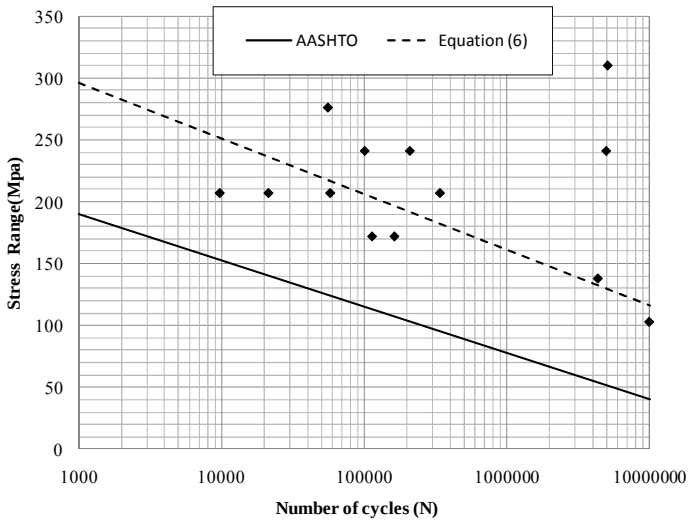


Figure 31 - Test results by Kwon and Engelhardt (2010)

3. Survey over residual strength tests

3.1. Tests by Oehlers

Oehlers (1990) reported on experimental tests to show that the monotonic strength of stud shear connectors is reduced under fatigue loads. This author established a method for the shear connection design that allows the reduction in the monotonic strength due to fatigue loads. A series of 14 identically manufactured push-out specimens were examined as shown in Figure 32. The studs were 12.7 mm in diameter and 75 mm in height. No information was given about the rebar.

The specimens were tested in three series, *S*, *F*, and *M*. The specimens in series *S* were tested to determine the static ultimate strength, P_u, of the shear connection. P_u is the mean value of the three results, experimentally determined as 54.3 kN. In series *F*, cyclic loading was continuously applied until the connector fractured. The range of the cyclic load, 0.25 P_u, was held constant and the peak of the cyclic load was varied. In series *M*, a block of cyclic loads was applied and then the specimens were loaded monotonically to failure; the range and peak of cyclic load was held constant and the number of cycles in a block varied. The test details and results are listed up in Table 16. P_s is the residual static strength per connector after certain cycling numbers.

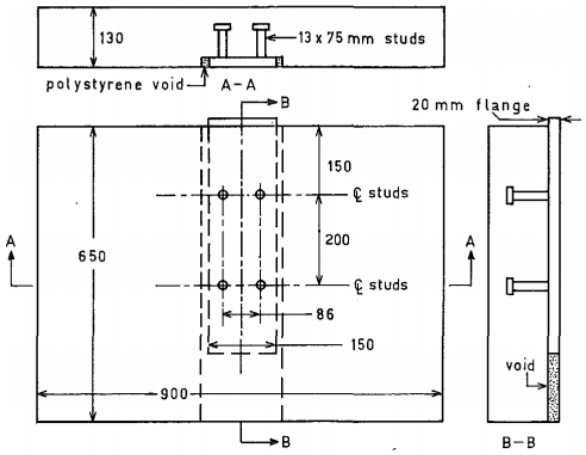


Figure 32 - Push-out-specimen used by Oehlers (1990)

Table 16 - Test results by Oehlers (1990)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta P/P_u$	N	P_s
		MPa	MPa	MPa	mm	kN	kN	kN		($\times 10^3$)	kN
1	S1	60	-	458.0	12.7	52.3	-	-		0	52.3
2	S2	60	-	458.0	12.7	56.0	-	-		0	56.0
3	S3	60	-	458.0	12.7	54.7	-	-		0	54.7
4	F1	60	-	458.0	12.7	44.9	31.3	13.6	0.25	572.0	-

5	F2	60	-	458.0	12.7	35.9	22.3	13.6	0.25	718.0	-
6	F3	60	-	458.0	12.7	26.9	13.3	13.6	0.25	1088.0	-
7	F4	60	-	458.0	12.7	26.9	13.3	13.6	0.25	895.0	-
8	F5	60	-	458.0	12.7	15.6	2.0	13.6	0.25	1251.0	-
9	F6	60	-	458.0	12.7	15.6	2.0	13.6	0.25	1507.0	-
10	M1	60	-	458.0	12.7	15.6	2.0	13.6	0.25	250.0	46.1
11	M2	60	-	458.0	12.7	15.6	2.0	13.6	0.25	500.0	43.6
12	M3	60	-	458.0	12.7	15.6	2.0	13.6	0.25	750.0	40.1
13	M4	60	-	458.0	12.7	15.6	2.0	13.6	0.25	1026.0	30.0
14	M5	60	-	458.0	12.7	15.6	2.0	13.6	0.25	1250.0	26.5

The three series results could be plotted in one diagram as shown in Figure 33. For series F, the P_{max} is taken as the longitudinal axis and for series M, the P_s is the longitudinal parameter. The relationship between the load and the cycling number is given by the following equation:

$$N = N_f(1-\frac{P_s}{P_u}) \quad (7)$$

The fatigue life N_f is a purely theoretical life that can only occur if monotonic failure is prevented, it is the fatigue life if a fatigue crack could propagate through the whole area of the stud without the occurrence of the bright monotonic failure zone. And N can be defined as the number of cycles of load to cause monotonic strength to reduce from P_u to P_s.

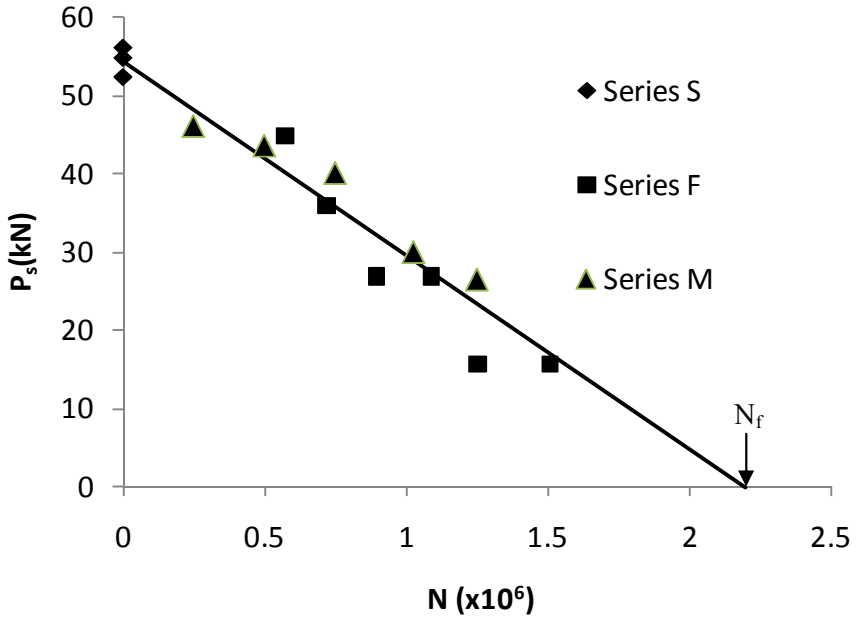


Figure 33 – Experimental interaction between strength and Endurance

3.2. Tests by Gattesco and Giuriani

Gattesco and Giuriani (1996) reported four stud connector push-out tests, two for monotonic loading and two for cyclic loading. The specimen details were shown in Figure 34. This special direct test was set up in order to remove some limitations of the normal push-out test. Here the stud shank diameter was 19 mm and overall height was 125 mm. The test details and results are listed up in Table 17.And the value of P_u derived from static tests is 106.1 kN.

The main arm of this test was to get the information on the shape of the load-slip curves and the damage accumulation at the end of each cycle for shear. This kind of fatigue behavior would be well investigated in next chapter.

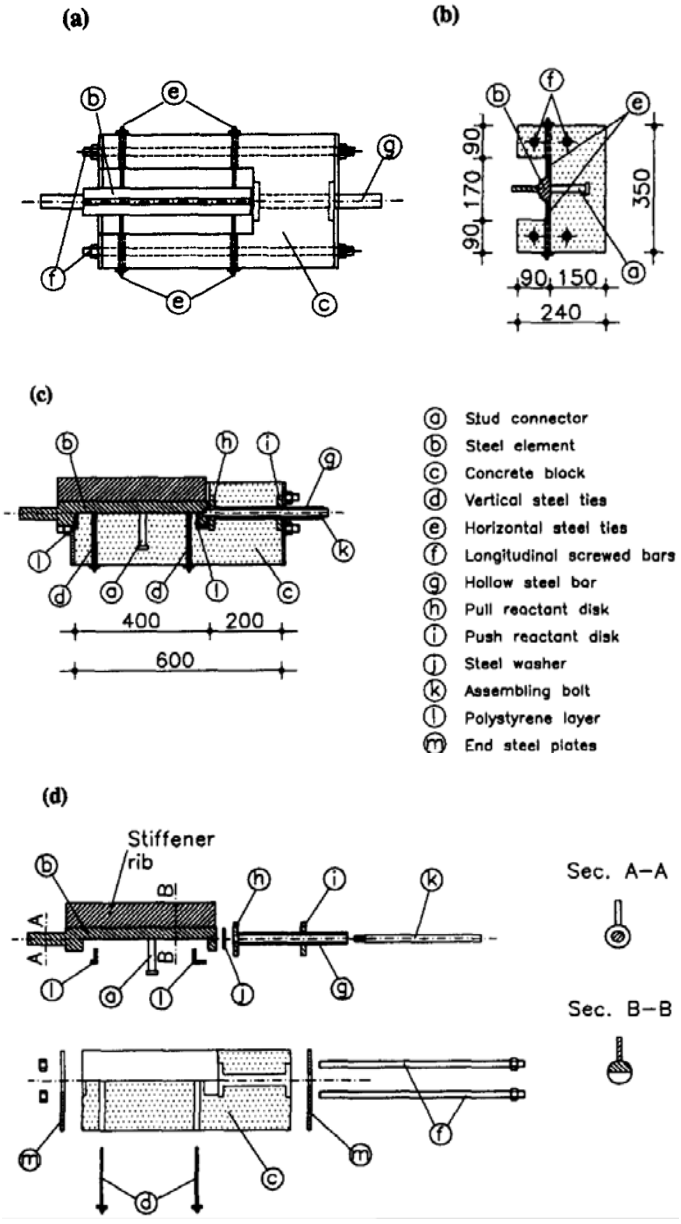


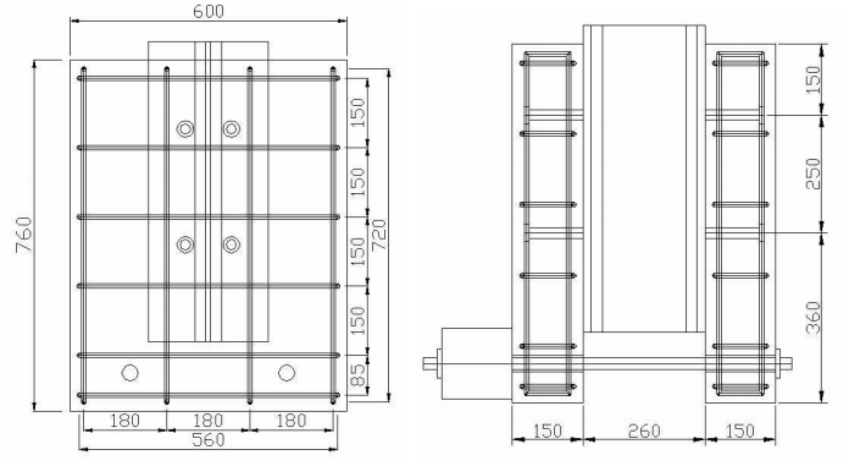
Figure 34 - specimen details used by Gattesco and Giuriani (1996): (a) plan of specimen; (b) transversal section; (c) longitudinal section; (d) exploded view

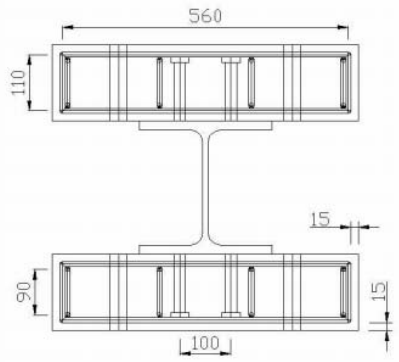
Table 17 - Test results of Gattesco and Giuriani (1996)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta P/P_u$	N	P_s
		MPa	MPa	MPa	mm	kN	kN	kN		($\times 10^3$)	kN
						30	11.8	18.2	0.17	10	
						40	11.8	28.2	0.27	11	
						50	9.4	40.6	0.38	28	
1	C1	32.5	350	480	19	60	7.2	52.8	0.50	54	99.4
						70	4.2	65.8	0.62	330	
						80	-1.2	81.2	0.77	650	
						80	-0.9	80.9	0.76	140	
						50	2.25	47.75	0.45	25	
2	C2	32.5	350	480	19	70	-2	72	0.68	500	96.1
						80	-9	89	0.84	100	

3.3. Tests by Bro and Westberg

Bro and Westberg (2004) reported experimental studies for steel stud shear connectors, using the EC4 standard push-out specimens as shown in Figure 35. The studs were 22 mm in diameter and 125 mm in height. The diameter of rebar was 10 mm. The push-out tests performed were five static, one endurance test and four residual strength tests. Here only fatigue tests results are listed up in Table 18. P_u derived from static results is 181kN. The static and the residual test results could be plotted in one diagram as Figure 36. Similar to the former results, the residual strength here has a linear regression too.





Reinforcement: ribbed bars ∅ 10 mm.

Steel section: HEB 260.

Figure 35 - Push-out-specimen used by Bro and Westberg (2004)

Table 18 - Test results by Bro and Westberg (2004)

	Test	f_c	f_y	f_u	d	P_{max}	P_{min}	ΔP	$\Delta P/P_u$	N	P_s
		MPa	MPa	MPa	mm	kN	kN	kN		($\times 10^3$)	kN
1	1	30	350.0	450.0	22	107.5	71.3	36.2	0.20	4900	-
2	1	30	350.0	450.0	22	107.5	71.3	36.2	0.20	400	166.1
3	2	30	350.0	450.0	22	107.5	71.3	36.2	0.20	1000	161.9
4	3	30	350.0	450.0	22	107.5	71.3	36.2	0.20	1200	159.6
5	4	30	350.0	450.0	22	107.5	71.3	36.2	0.20	2000	164.1

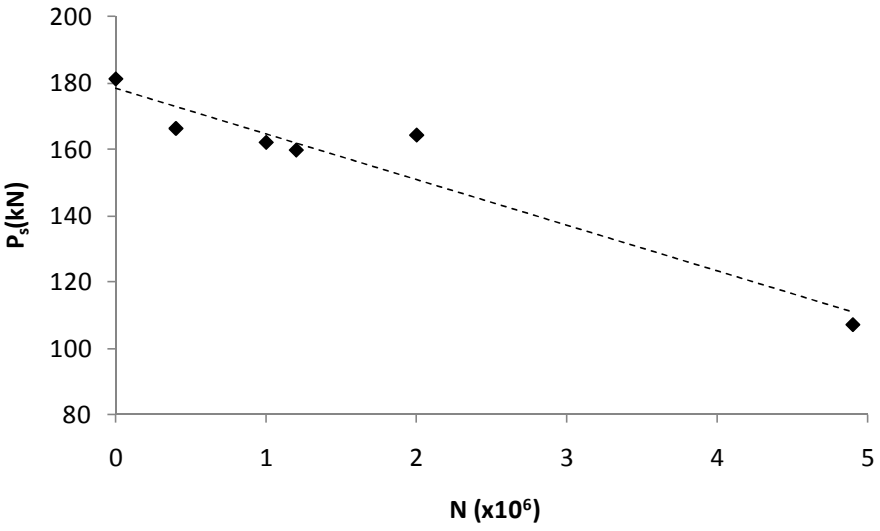


Figure 36 - Test results by Bro and Westberg (2004)

3.4. Tests by Ahn and Kim with stud connectors

Ahn and Kim (2007) also reported three residual strength tests for a new bridge deck system using the identical specimens as shown in Figure 27. Here only the test details and results are listed up in Table 19. P_u derived from static results is 100.5 kN.The residual strength tests results by Ahn and Kim together with part endurance tests results in chapter 2.13 are plotted in

Figure 37. There is a linear interpolation between these results.

Table 19 - Test results by Ahn and Kim (2007)

	Test	f_c MPa	f_y MPa	f_u MPa	d mm	P_{max} kN	P_{min} kN	ΔP kN	$\Delta P/P_u$	N ($\times 10^3$)	P_s kN
1	ST-R-A1	30.0	351.0	422.0	16	20.1	0.0	20.1	0.20	500	89.6
2	R1	30.0	351.0	422.0	16	20.1	0.0	20.1	0.20	1000	86.3
3	R2	30.0	351.0	422.0	16	20.1	0.0	20.1	0.20	1500	77.6

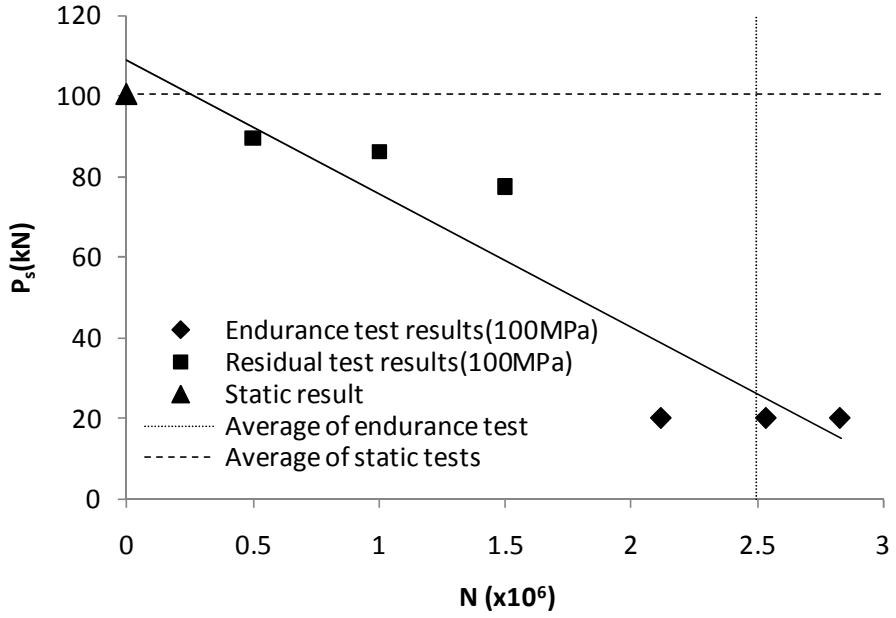


Figure 37 - Test results by Ahn and Kim (2007)

3.5. Tests by Hanswille and Porsch

Hanswille and Porsch (2007) reported a series of experimental work with standard EC4 push-out specimens as shown in Figure 38, to determine the fatigue life and a possible reduction of the static strength of the headed shear studs subjected to unidirectional cyclic loading. The studs were 22 mm in diameter and 125 mm in height. Standard Bent bars with diameters of 10 mm and 12 mm were used in the concrete slabs as reinforcement.

72 specimens were prepared and grouped in 6 series. S1~S4 and S5E deal with constant amplitude tests. The varying loading parameters of these series are the peak load P_{max} and the loading range ΔP. In each series, three static tests were initially performed to determine the mean value of the ultimate static load P_u of the push-out specimen and three load controlled cyclic tests were performed to determine the mean fatigue life N of the push-out specimens. Subsequently, three cyclic tests were conducted for approximately 30% of the mean fatigue life N and three more tests were conducted for approximately 70% of the mean fatigue life N. After

reaching the pre-defined number of cycles, each of these six test specimens was statically loaded until failure, under displacement control, to obtain the reduced static strength after high cycle preloading.

Specimens of series S5 and S6 were tested with two and four blocks of loading sequences. These tests were performed with a constant loading range, in which the peak load was increased or decreased subsequently while the loading range was held constant, as shown in Figure 39.

The specimen numbers of each series and the static strength are listed up in Table 20.

The test details and results of S1~S4 and S5~S6 are listed up separately in Table 21 and Table 22.

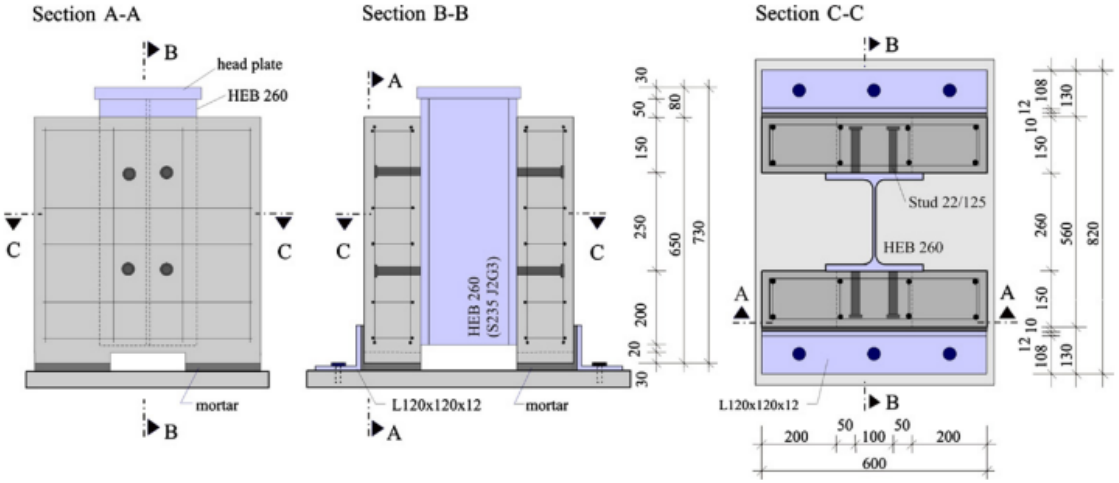


Figure 38 - Push-out-specimen used by Hanswille and Porsch (2007)

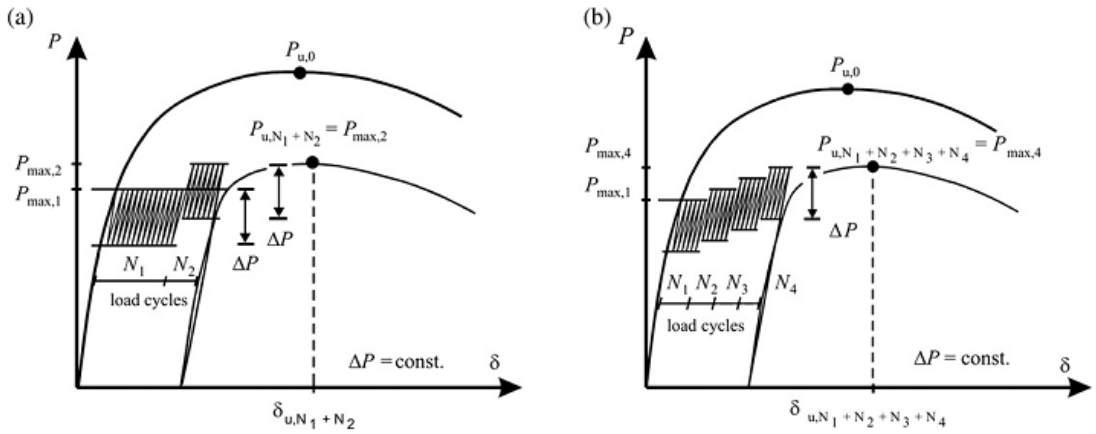


Figure 39 - Tests with multiple block of loading (Hanswille and Porsch 2007)

Table 20 - Number of specimens tested in each series and static ultimate strength

Series		S1	S2	S3	S4	S5E	S5-2	S5-3	S5-4	S5-6	S6-3	S6-4
Number Of tests	P_u	3	3	3	3	3	3	1	4	4	3	3
	N	3	3	3	3	1						
	$0.3N$	3	3	3	3	1						
	$0.7N$	3	3	3	3	1						
P_u (kN)		205	184	201	181	189	186				196	

Table 21 - Test results by Hanswille and Porsch (S1~S4)

	Test	f_c MPa	f_y MPa	f_u MPa	d mm	P_{max} kN	P_{min} kN	ΔP kN	$\Delta P/P_u$	N ($\times 10^3$)	P_s kN
1			337.0	448.0	22	90.2	49.2	41.0	0.20	6200	-
2	S1	44-52	337.0	448.0	22	90.2	49.2	41.0	0.20	1984	154
3			337.0	448.0	22	90.2	49.2	41.0	0.20	5580	129
4			337.0	448.0	22	130.6	84.6	46.0	0.25	1200	-
5	S2	42-45	337.0	448.0	22	130.6	84.6	46.0	0.25	384	174
6			337.0	448.0	22	130.6	84.6	46.0	0.25	840	154
7			337.0	448.0	22	88.4	38.1	50.3	0.25	5100	-
8	S3	53-56	337.0	448.0	22	88.4	38.1	50.3	0.25	1224	133
9			337.0	448.0	22	88.4	38.1	50.3	0.25	3519	123
10			337.0	448.0	22	128.5	92.3	36.2	0.20	3500	-
11	S4	43	337.0	448.0	22	128.5	92.3	36.2	0.20	1015	181
12			337.0	448.0	22	128.5	92.3	36.2	0.20	2520	156
13			337.0	448.0	22	56.7	9.4	47.3	0.25	6400	-
14	S5E	43	337.0	448.0	22	56.7	9.4	47.3	0.25	3776	111
15			337.0	448.0	22	56.7	9.4	47.3	0.25	4672	114

Table 22 - Test results by Hanswille and Porsch (S5~S6)

	Test	f_c MPa	f_y MPa	f_u MPa	d mm	P_{max} kN	P_{min} kN	ΔP kN	$\Delta P/P_u$	N ($\times 10^3$)	P_s kN
1	S5-2a	43	337.0	448.0	22	133	86	47	0.25	204	-
						83	36	47	0.25	792	-
2	S5-2c	43	337.0	448.0	22	133	86	47	0.25	198	-
						83	36	47	0.25	1440	-
3	S5-3a	43	337.0	448.0	22	83	36	47	0.25	1099	-

						133	86	47	0.25	-	-
4	S5-4a	43	337.0	448.0	22	83	36	47	0.25	473	-
						56	9	47	0.25	1365	-
5	S5-4b	43	337.0	448.0	22	83	36	47	0.25	517	-
						56	9	47	0.25	772	-
6	S5-4c	43	337.0	448.0	22	83	36	47	0.25	544	-
						56	9	47	0.25	735	-
7	S5-4d	43	337.0	448.0	22	83	36	47	0.25	542	-
						56	9	47	0.25	3396	-
8	S5-6a	43	337.0	448.0	22	56	9	47	0.25	537	-
						83	36	47	0.25	5821	-
9	S5-6b	43	337.0	448.0	22	56	9	47	0.25	1223	-
						83	36	47	0.25	761	-
10	S5-6c	43	337.0	448.0	22	56	9	47	0.25	1295	-
						83	36	47	0.25	1744	-
11	S5-6d	43	337.0	448.0	22	56	9	47	0.25	1277	-
						83	36	47	0.25	3206	-
12	S6-3a	46	337.0	448.0	22	83	45	38	0.20	756	-
						101	63	38	0.20	768	-
						120	82	38	0.20	770	-
						139	101	38	0.20	868	-
13	S6-3b	46	337.0	448.0	22	83	45	38	0.20	765	-
						101	63	38	0.20	804	-
						120	82	38	0.20	785	-
						139	101	38	0.20	324	-
14	S6-3c	46	337.0	448.0	22	83	45	38	0.20	754	-
						101	63	38	0.20	759	-
						120	82	38	0.20	750	-
						139	101	38	0.20	449	-
15	S6-4a	46	337.0	448.0	22	139	101	38	0.20	550	-
						120	82	38	0.20	763	-
						101	63	38	0.20	754	-
						83	45	38	0.20	583	-
16	S6-4b	46	337.0	448.0	22	139	101	38	0.20	550	-
						120	82	38	0.20	758	-
						101	63	38	0.20	750	-

						83	45	38	0.20	756	-
17	S6-4c	46	337.0	448.0	22	139	101	38	0.20	540	-
						120	82	38	0.20	753	-
						101	63	38	0.20	753	-
						83	45	38	0.20	1208	-

Based on such massive test results, several characteristics have been found by authors. First, in Figure 40 the results of the constant amplitude tests of series S1- S5E are compared with the corresponding test results, from which the fatigue strength curve in Eurocode 4 was derived. The test results are in good agreement with the given prediction according to Eurocode 4.

Second, in Figure 41, where the results are related to the mean static strength and the mean fatigue life of each series respectively, it is shown that the influence of the cyclic loading becomes evident. Especially in series S1, S3 and S5E with low peak loads, the rapid increase of the static strength within the first 20% of the fatigue life is noteworthy. On the other hand N is much greater for the series with high peak loads. The test results also indicate an early crack initiation at approximately 10%-20% of the fatigue life which causes the reduction of the static strength. Constant amplitude tests have shown that the magnitude of the peak load P_{max} of the cycle loading has a significant effect on the way cracks are formed at the stud foot.

Third, evaluation of the tests with multiple blocks of loading on the basis of linear damage accumulation hypothesis of Palmgren and Miner, on which the present design codes rely, is shown in Figure 42. It is obvious that expect for one test, all results of the lifetime prediction according to Palmgren and Miner lie on the unsafe side.

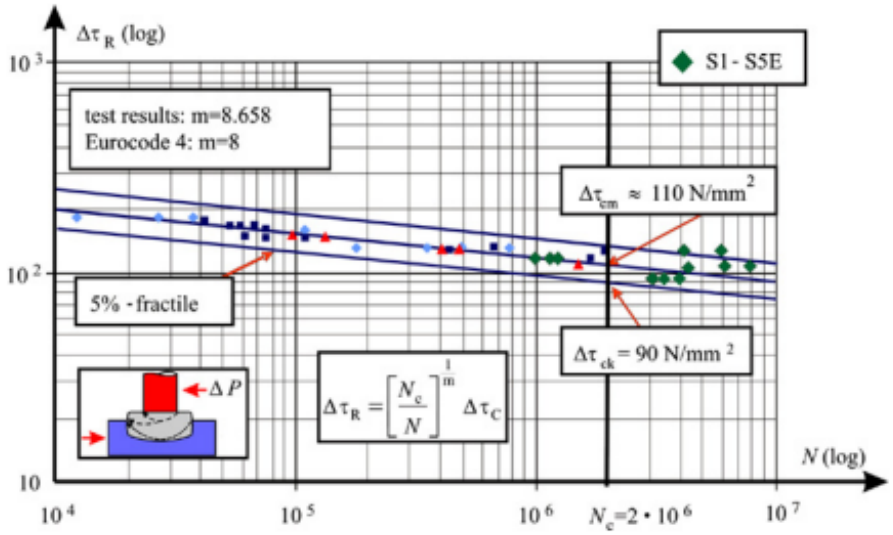


Figure 40 – Comparison of fatigue test results with the prediction in Eurocode 4

(Hanswille and Porsch, 2007)

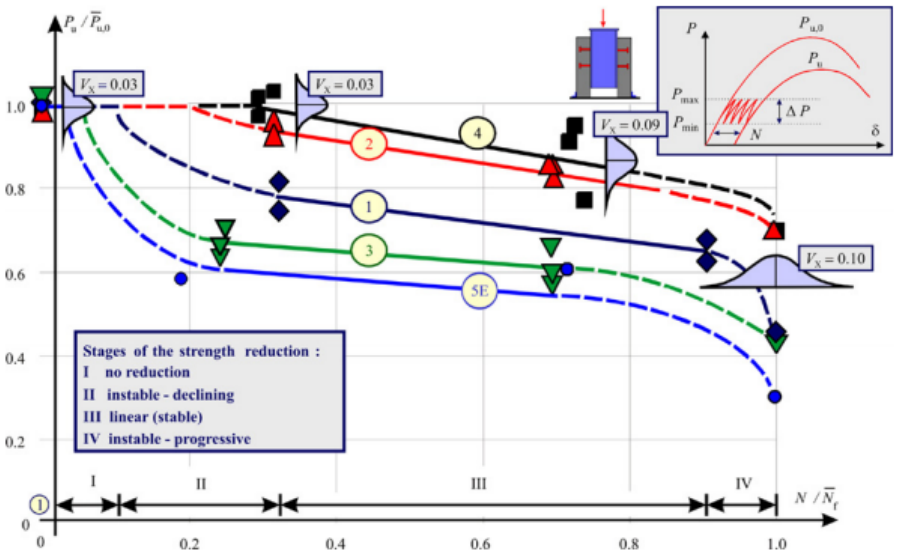


Figure 41 – Decrease of static strength versus lifetime due to high cycling loading (Hanswille and Porsch, 2007)

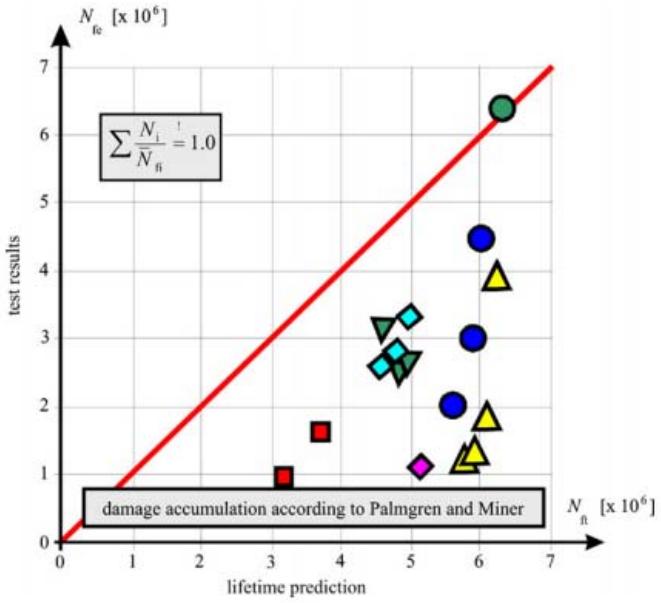


Figure 42 – Comparison between the test results with the results of the lifetime prediction to Palmgren-Miner (Hanswille and Porsch, 2007)

3.6. Tests by Ahn and Kim with Perfobond connectors

Ahn and Kim (2008) reported 11 push-out tests for Perfobond rib shear connectors under cyclic loads. Standard EC4 push-put specimens were examined and one main parameter was considered, the existence or absence of rebars. The diameter of rebar was not distinct. The details of Perfobond shear connectors and the push out specimens (with rebar) are shown in Figure 43.

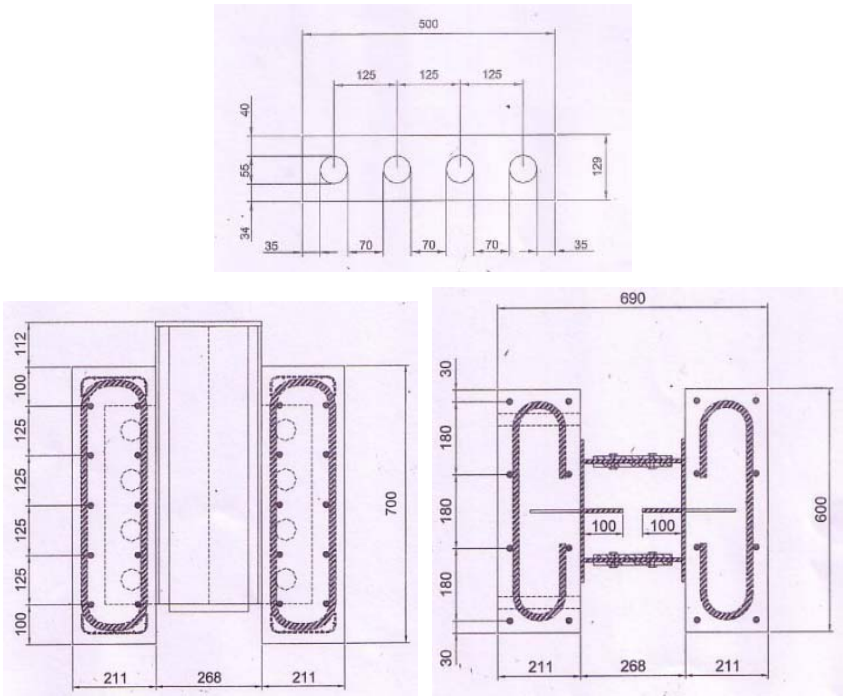


Figure 43 - Connector shape and push-out-specimen used by Ahn and Kim (2008)

The test details and results are listed up in Table 23. The specimens of Series A and B had no rebars and the specimens of series C had rebars. The mean ultimate strength Pu derived from static results, are 628.3 kN for series A and B, and 1268.2 kN for series C. The test results are plotted in Figure 44. From the tests results, it was found that the residual shear capacity of specimens without transverse rebars decreased to about 65% that of static shear capacity. In the case of specimens with transverse rebars in their holes, however, their residual shear capacity was the same as their static shear capacity.

Table 23 - Test results of Perfobond connectors by Ahn and Kim (2008)

	Test	f_c MPa	f_y MPa	f_u MPa	d mm	P_{max} kN	P_{min} kN	ΔP kN	$\Delta P/P_u$	N ($\times 10^3$)	P_s kN
1	PF-R-A1	30	356	440.0	-	56.1	1.3	54.8	0.09	75	387.4
2	PF-R-A2	30	356	440.0	-	56.1	1.3	54.8	0.09	100	417.6
3	PF-R-A3	30	356	440.0	-	56.1	1.3	54.8	0.09	125	415.0
4	PF-R-B1	30	356	440.0	-	28.7	1.3	27.4	0.05	75	412.1
5	PF-R-B2	30	356	440.0	-	28.7	1.3	27.4	0.05	100	420.0
6	PF-R-B3	30	356	440.0	-	28.7	1.3	27.4	0.05	125	434.0
7	PF-R-C1	30	356	440.0	-	56.1	1.3	54.8	0.09	75	1270.2
8	PF-R-C2	30	356	440.0	-	56.1	1.3	54.8	0.09	100	1238.3
9	PF-R-C3	30	356	440.0	-	56.1	1.3	54.8	0.09	125	1221.0
10	PF-R-C4	30	356	440.0	-	56.1	1.3	54.8	0.09	1000	1299.8
11	PF-R-C5	30	356	440.0	-	56.1	1.3	54.8	0.09	2000	1310.9

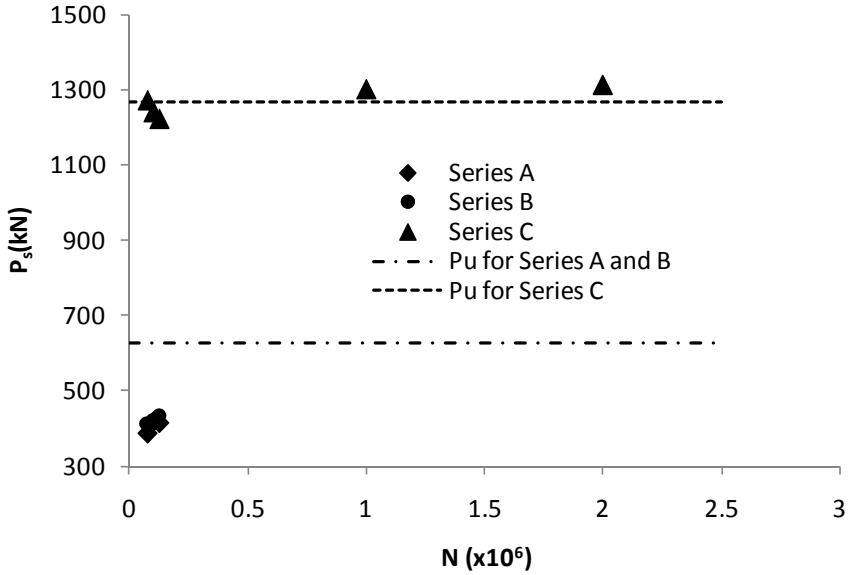


Figure 44 –Test results of Perfobond connectors by Ahn and Kim

4. Survey over slip tests

4.1. Tests by Hallam

Although Hallam (1976) reported 13 push-out tests conducted under constant amplitude unidirectional repeated loading as shown in part 2.4, the quantitative expressions for the slip growth per cycle, as a function of the load range were developed. The load-slip data from Hallam are listed up in Table 24. The calculation of static strength P_u was derived from CP 117-Composite Construction in structural Steel and Concrete (BSI 1967).Here δ presents the slip per cycle and the unit is mm.

Table 24 - Test results of stud connectors by Hallam (1976)

	Test	f_c	P_{max}	P_{min}	ΔP	$\Delta P/P_u$	N	δ/1000
		MPa	kN	kN	kN		(x10^3)	μm
1	PS4	35.2	49.9	2.3	47.6	0.41	1303.0	20.3
2	PS5	35.2	49.9	2.3	47.6	0.41	824.0	19.1
3	PS6	33.2	28.4	2.3	26.1	0.24	845.0	0.178
4	PS7	33.2	28.4	2.3	26.1	0.24	652.0	0.0356
5	PS8	32.8	23.2	2.3	21.0	0.20	652.0	0.0483
6	PS9	32.8	23.2	2.3	21.0	0.20	52.8	16.4
7	PS10	30.3	44.2	2.3	44.2	0.39	52.8	5.18
8	PS11	30.3	44.2	2.3	44.2	0.39	58.6	5.15
9	PS12	22.8	33.7	2.3	33.7	0.33	67.9	7
10	PS13	22.8	33.7	2.3	33.7	0.33	3170.0	20.3

4.2. Tests by Gattesco and Giuriani

Gattesco and Giuriani (1997) reported 8 direct shear tests with 19 mm diameter shear studs under low-cycle fatigue, to determine the low-cycle fatigue endurance of shear connectors, using a displacement control approach where they determined the fatigue life of a connector with a given slip history. The specimen details were already shown in Figure 34. The test details and the maximum induced slip values and corresponding number of cycles to failure are listed in Table 25.

The authors found that as the maximum slip value exceeded 1 mm, the fatigue life of the connectors was lower than 10,000 cycles. The corresponding shear load at every displacement cycle was also found to reduce at the beginning of each test due to concrete damage around the shear connector. Fatigue failure was observed through the stud shank.

Table 25 - Tests result by Gattesco and Giuriani (1997)

	Test	f_c	f_y	f_u	d	S_{max}	S_{min}	N
		MPa	MPa	MPa	mm	mm	mm	(x10^3)
1	1	39	350	480	19	0.80	0.40	38338
2	2	39	350	480	19	1.00	0.50	18400
3	3	39	350	480	19	1.00	0.50	13200
4	4	39	350	480	19	1.25	0.63	5274
5	5	39	350	480	19	1.50	0.75	3040
6	6	39	350	480	19	2.00	1.00	3230
7	7	39	350	480	19	2.00	1.00	1440
8	8	39	350	480	19	3.00	1.50	432

4.3. Tests by Taplin

In order to measure the incremental slip of stud shear connectors under repeated loading, Taplin (1997) conducted a series push-out tests with stud of 12.5 mm diameter and 50 mm height. The studs had a specified minimum ultimate tensile strength fu of 410 MPa, yield strength f_y of 345 MPa. Transverse reinforcement, comprising two layers of 8 mm diameter wires with yield strength of 450 MPa was cast into the slab to prevent longitudinal shear failure, as shown in Figure 45.

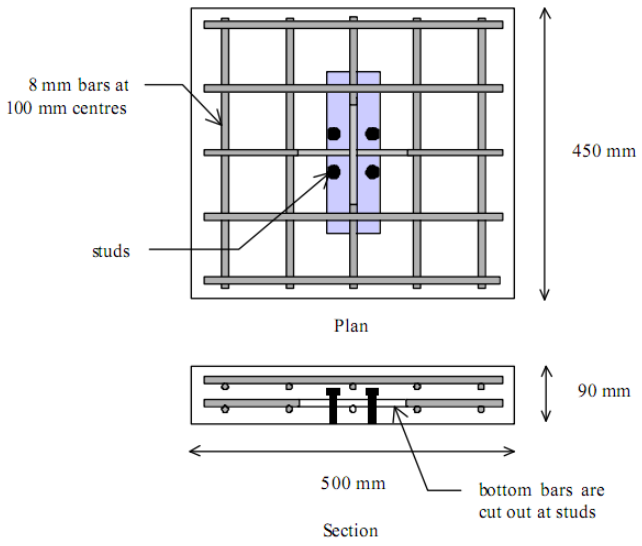


Figure 45 - Push-out specimens used by Taplin (1997)

A total of 15 push-out specimens were tested, comprising 4 monotonic tests to determine the static strength of the stud shear connectors, 7 tests with symmetric cyclic loading, and 4 tests with unidirectional cyclic loading. All the cyclic tests were performed with predefined load ranges. The load was applied for 30 cycles at each load range. The rate of slip growth for all symmetric cyclic tests is listed up in Table 26 and the rate of slip growth for all unidirectional cyclic tests is listed up in Table 27. Here according to the results of monotonic tests, the calculated ultimate strength P_u is 50kN.

Table 26 - Rate of slip growth for all symmetric cyclic tests

	Test	f_c	P_{max}	P_{min}	ΔP	$\Delta P/P_u$	N	δ/1000(μm)			
		MPa	kN	kN	kN		(x10^3)	Lvdt 1		Lvdt 2	
								+	-	+	-
1	4	-	6.25	-6.25	12.50	0.25	0.03	0.94	-0.12	1.24	-0.89
			9.38	-9.38	18.75	0.375	0.03	3.31	1.32	2.59	-2.65
			12.50	-12.50	25.00	0.5	0.03	8.36	3.53	2.28	0.61
			15.63	-15.63	31.25	0.625	0.03	48.6	8.99	23.9	7.59
2	5	47.4	6.25	-6.25	12.50	0.25	0.03	1.42	0.38	-0.27	1.24
			9.38	-9.38	18.75	0.375	0.03	2.67	-0.62	1.87	0.31
			11.25	-11.25	22.50	0.45	0.03	6.48		2.31	0.67
			12.50	-12.50	25.00	0.5	0.03	16.8	2.02	9.67	
			13.75	-13.75	27.50	0.55	0.03		9.75	42.4	14.5
3	6	43.1	6.25	-6.25	12.50	0.25	0.03	1.22	0.22	0.53	0.94
			9.38	-9.38	18.75	0.375	0.03	2.70	0.90	3.25	1.38
			11.25	-11.25	22.50	0.45	0.03	12.1	2.09		6.33

			6.25	-6.25	12.50	0.25	0.03	0.48	0.14	0.74	0.58
			9.38	-9.38	18.75	0.375	0.03	0.67	0.30	0.60	1.45
4	11	48.0	11.25	-11.25	22.50	0.45	0.03	2.23	1.19	2.02	1.01
			12.50	-12.50	25.00	0.5	0.03	5.89	4.05	3.36	0.42
			14.38	-14.38	28.75	0.575	0.03	21.7	23.7	14.2	4.57
			6.25	-6.25	12.50	0.25	0.03	-0.23	1.19	1.19	0.10
			8.13	-8.13	16.25	0.325	0.03	0.42	1.27	1.56	-0.43
5	12	46.7	10.63	-10.63	21.25	0.425	0.03	1.87	3.60	1.79	-0.25
			11.88	-11.88	23.75	0.475	0.03	0.54	4.86	1.65	0.41
			13.13	-13.13	26.25	0.525	0.03	3.89	3.57	4.46	0.38
			6.25	-6.25	12.50	0.25	0.03	0.88	0.32	-0.51	-1.24
			8.13	-8.13	16.25	0.325	0.03	1.74	1.39	-0.54	0.21
6	15	44.5	10.63	-10.63	21.25	0.425	0.03	1.66	2.53	4.03	0.35
			11.88	-11.88	23.75	0.475	0.03	6.62	2.13	5.93	0.71
			13.13	-13.13	26.25	0.525	0.03	26.5	3.78	20.5	3.89
			6.25	-6.25	12.50	0.25	0.03	-0.29	1.79	-0.43	1.22
			8.13	-8.13	16.25	0.325	0.03	0.08	1.26	0.15	0.94
			9.38	-9.38	18.75	0.375	0.03	-0.29	1.62	-0.13	1.28
			10.63	-10.63	21.25	0.425	0.03	0.27	1.25	-0.01	1.38
7	16	61.7	11.88	-11.88	23.75	0.475	0.03	0.46	1.47	0.68	1.48
			13.13	-13.13	26.25	0.525	0.03	1.72	2.66	1.95	2.01
			14.38	-14.38	28.75	0.575	0.03	5.86	5.12	2.61	5.42
			15.63	-15.63	31.25	0.625	0.03	60.3	69.5	4.30	20.2

Table 27 - Rate of slip growth for all unidirectional cyclic tests

	Test	f_c	P_{max}	P_{min}	ΔP	$\Delta P/P_u$	N	δ/1000(μm)	
		MPa	kN	kN	kN		(x10^3)	Lvdt 1	Lvdt 2
			15.00	1.25	13.75	0.275	0.03	0.37	0.56
			21.25	1.25	20.00	0.4	0.03	0.93	1.05
			25.00	1.25	23.75	0.475	0.03	1.33	1.60
			27.50	1.25	26.25	0.525	0.03	2.37	2.52
1	9	50.0	30.00	1.25	28.75	0.575	0.03	3.49	3.55
			33.75	1.25	32.50	0.65	0.03	8.84	7.52
			36.25	1.25	35.00	0.7	0.03	23.4	14.5
			40.00	1.25	38.75	0.775	0.03	211	184

2	14	44.5	15.00	1.25	13.75	0.275	0.03	0.41	0.74
			21.25	1.25	20.00	0.4	0.03	1.30	1.55
			25.00	1.25	23.75	0.475	0.03	1.60	2.02
			27.50	1.25	26.25	0.525	0.03	2.11	2.54
			30.00	1.25	28.75	0.575	0.03	3.12	4.44
			33.75	1.25	32.50	0.65	0.03	9.81	20.6
			36.25	1.25	35.00	0.7	0.03	23.4	52.7
			40.00	1.25	38.75	0.775	0.03	145	333
3	13	45.3	15.00	1.25	13.75	0.275	0.03	0.10	
			21.25	1.25	20.00	0.4	0.03	0.78	
			25.00	1.25	23.75	0.475	0.03	1.42	
			27.50	1.25	26.25	0.525	0.03	1.72	
			30.00	1.25	28.75	0.575	0.03	2.29	2.53
			33.75	1.25	32.50	0.65	0.03	9.17	8.03
			36.25	1.25	35.00	0.7	0.03	14.1	18.6
			40.00	1.25	38.75	0.775	0.03	48.3	54.0
			42.50	1.25	41.25	0.825	0.03	59.9	56.0
			46.25	1.25	45.00	0.9	0.03	233	300
4	17	60.5	15.00	1.25	13.75	0.275	0.03	0.06	0.03
			21.25	1.25	20.00	0.4	0.03	0.50	0.08
			25.00	1.25	23.75	0.475	0.03	1.24	0.56
			27.50	1.25	26.25	0.525	0.03	1.41	0.24
			30.00	1.25	28.75	0.575	0.03	2.91	0.43
			33.75	1.25	32.50	0.65	0.03	3.20	0.92
			36.25	1.25	35.00	0.7	0.03	2.82	1.12
			40.00	1.25	38.75	0.775	0.03	3.95	1.50
			42.50	1.25	41.25	0.825	0.03	5.24	2.00
			46.25	1.25	45.00	0.9	0.03	11.4	4.23
			48.75	1.25	47.50	0.95	0.03	17.6	5.76

4.4. Tests by Valente

Valente (2007) conducted 2 static and 4 cyclic standard push-out tests with headed studs in high strength lightweight concrete solid slabs. The studs had 13 mm diameter and 50 mm height. And the ultimate tensile strength, f_u, and yelding tensile strength, f_y, were 501 MPa and 596 MPa, respectively. Two layers of 5 mm diameter bars were disposed as shown in Figure 46. The cyclic test details and results are listed up in Table 28. The ultimate strength P_u here is 58kN.

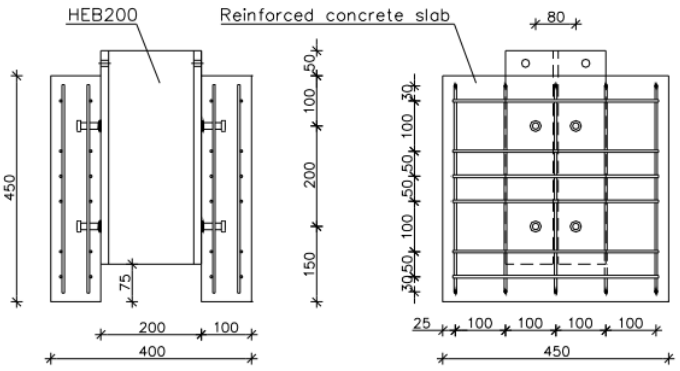


Figure 46 - Push-out specimens used by Valente (2007)

Table 28 - Slip results by Valente (2007)

	Test	f_c	P_{max}	P_{min}	ΔP	$\Delta P/P_u$	N	P_s	δ/1000
		MPa	kN	kN	kN		(x10^3)	kN	μm
1	CN13.3	63.79	26.7	5.1	21.6	0.38	10	58.0	3.34
2	CN13.4	63.34	31.9	4.9	27.0	0.47	16	57.7	5.33
3	CN13.5	63.79	12.4	3.7	8.7	0.15	1	58.1	0.49
			24.3	3.4	21.3	0.37	1		7.38
			30.6	3.5	27.1	0.47	1		20.12
			36.5	3.2	33.3	0.58	1		59.24
			40.3	3.2	37.1	0.65	1		143.0
4	CN13.6	56.65	16.1	4.8	11.3	0.21	1	48.2	5.92
			31.9	4.5	27.4	0.51	1		69.27
			40.0	4.6	35.4	0.65	1		469.23
			48.2	4.5	43.7	0.81	1		28483

5. Discussion

5.1. Fatigue life calculation for studs

From the previous chapters' description, it is clear that most of the research work for fatigue behavior of shear connector is dealt with shear studs. These studies focused on endurance tests, residual strength tests, low-cycle fatigue or slip behavior. In addition, Slutter and Fisher (1966) researched the fatigue strength of channel connectors, Ann and Kim (2008) conducted residual strength research on Perfobond shear connectors and Kwon and Engelhardt (2010) researched the fatigue strength of post-installed shear connectors.

Because there are more research results of studs, the fatigue behavior on this kind of shear connector is better studied, and the fatigue strength design criterions for shear studs are

established in international codes, such as Eurocode3 and 4, British code BS5400 and American code AASHTO. Johnson (2000) reanalyzed several results obtained in fatigue tests performed with studs and three distinct models for design were presented, the "EC model", the "BS model" and the "Peak load model".

The "EC model" as shown in equation (8), takes the stress range of shear studs as the main decisive factor for fatigue strength, no account of the static shear strength of the stud P_u or the maximum shear force applied P_{max}. It is applied now in Eurocode3 and 4 and AASHTO.

$$N(\Delta\tau)^m = 10^k \tag{8}$$

The "BS model" used in the British Bridge Code BS5400 takes in account the stud static strength as Equation (9) .

$$N(\Delta P / P_u)^m = 10^k \tag{9}$$

The "Peak Load model" as shown in equation (10) is based on the work developed by Oehlers (1990). Unlike other authors, it takes account on the maximum shear force P_{max}.

$$N_f(\Delta P / P_u)^m = 10^k \tag{10}$$

, where the endurance is $N = N_f[1-(P_{max}/P_u)]$

Table 29 lists up the comparison of these three models.

Table 29 - Design models for the fatigue life of shear studs

Calculation Model	Basic equation	Applicable equation
EC Model	$N(\Delta\tau)^m = 10^k$	$\lg N + 8\lg\Delta\tau = 21.93$ (Eurocode4) $\Delta\tau = \frac{4}{\pi}(238 - 29.51\lg N)$ (AASHTO)
BS Model	$N(\Delta P / P_u)^m = 10^k$	$\lg N + 8\lg(\Delta P / P_u) = 1.29$ (BS 5400)
Peak load Model	$N_f(\Delta P / P_u)^m = 10^k$, where $N = N_f[1-(P_{max}/P_u)]$	$P_{max}/P_u \le 0.6$ (applied condition)

Furthermore, Hanswille and Porsch (2007) selected and reanalyzed 65 fatigue resistance test results since 1960s, considering the selection criteria as:

- The specimen must consist of two concrete slabs and a steel beam with headed shear studs as shear connectors in according with Eurocode 4.
- The concrete slabs must be cast in a horizontal position.
- The stud must be welded with an adequate welding procedure ensuring the formation of a weld collar in accordance with EN 13918 and EN 14555.

- Test must be made with sufficient transverse reinforcement.
- Only test with unidirectional loading are considered.
- Only headed shear studs in solid slabs are considered.

Then the fatigue life of headed studs in push-put specimen subjected to unidirectional cyclic loading can be predicted with the following form:

$$\log N_f = \frac{1 - \frac{P_{\max}}{P_u}}{K_1 - K_2 \cdot \frac{P_{\max} - \frac{\Delta P}{2}}{P_u}} \tag{11}$$

$K_1$and K_2 are coefficients. For push-out specimens with lateral restraints of concrete slab, they could be taken as $K_1 = 0.1267$ $K_2 = 0.1344$, and for push-out specimens without lateral restraints, they are $K_1 = 0.1483$ $K_2 = 0.1680$.

If the static tests do not exist, the static strength of the headed shear studs P_u should be calculated with the model given in Eurocode 4. In EC4 the static strength is defined by the minimum of the two equations where the first one corresponds to the case “steel failure” and the other one to the case “concrete failure”:

$$P_{u1} = 0.37 \cdot d^2 \sqrt{E_{cm} \cdot f_c} \tag{12}$$

$$P_{u2} = f_u \cdot \frac{1}{4} \pi d^2 \tag{13}$$

In most of the tests the modulus of elasticity of concrete was not determined experimentally so this is determined on the basis of the relations given EC2 with the following equation:

$$E_{cm} = 9500 \cdot (f_c)^{1/3} \tag{14}$$

To illustrate the quality of the prediction on the relationship between the experimental fatigue life N_e and the theoretical fatigue life N_f is given in Figure 47 by Hanswille and Porsch (2007). Although it seems to have a good quality of prediction, because of the strict selection criteria for test results to be analyzed, the applicability of the equation (11) is limited. It could not be available for all the test specimens.

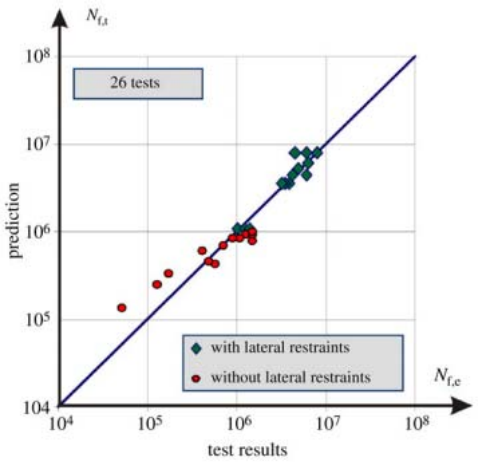


Figure 47 – Relationship between the experimental and theoretical fatigue life(Hanswille and Porsch, 2007)

5.2. Residual strength calculation for studs

There are two different views about the calculation models for the static residual strength of studs after a certain number of cycling loadings. One theory is that the residual strength has a linear regression according to the fatigue life as shown in Figure 33, Figure 36 and Figure 37, derived by the tests results of Oehlers (1990) , Bro and Westberg (2004) and Ahn and Kim (2007). Another is that the relationship between the residual strength and the fatigue life is nonlinear as shown in Figure 41, derived by the test results of Hanswille and Porsch (2007).

According to equation (7) established by Oehlers (1990), the following equation is used by Bro and Westberg (2004) to derive the residual strength:

$$P_s = P_u\left(1-\frac{N}{N_f}\right) \tag{15}$$

where $N_f = 12.9\times10^6$ for $\Delta P = 0.2P_u$ and $d = 22mm$.

Hanswille and Porsch (2007) established the following equation taking into account the limitations to predict the reduced satic strength at a given number of loading cycles:

$$\frac{P_s}{P_u} = 0.74\cdot\frac{P_{\max}}{P_u}\cdot\left(1-\frac{\Delta P}{P_{\max}}\right)+0.54-0.04\cdot\ln\left(\frac{N}{10^{\frac{1-\frac{P_{\max}}{P_u}}{0.1267-0.1344\cdot\frac{P_{\max}}{P_u}\cdot\left(1-\frac{\Delta P}{2P_{\max}}\right)}}-N}\right) \tag{16}$$

$$\begin{cases} \le 1 \\ \ge \dfrac{P_{\max}}{P_u} \end{cases}$$

To illustrate the quality of prediction of these two equations, the Comparison of the experimental and theoretical values of the reduced static strength by each equation is given in Figure 48 and Figure 49.It seems that both equations have a precision, equation (16) has less limitation for application, despite of the complicated formula.

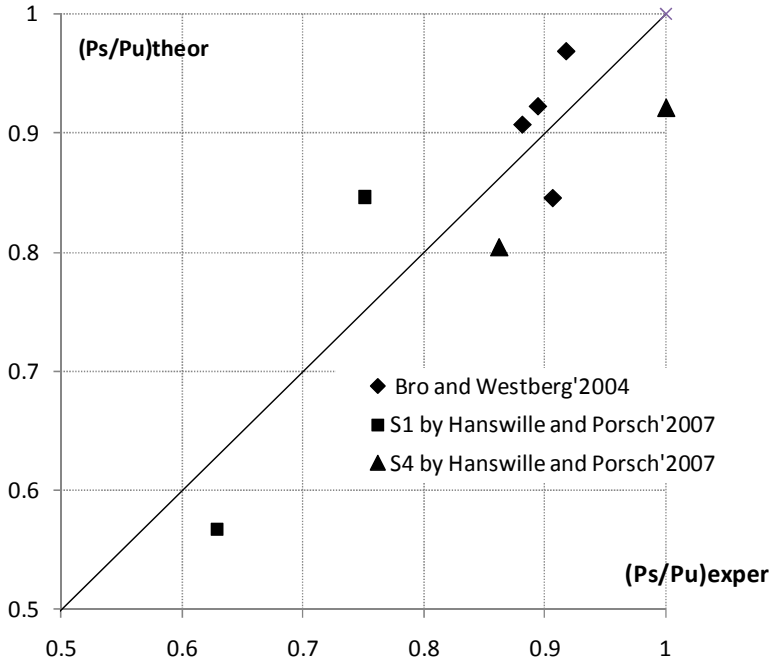


Figure 48 – Comparison of the experimental and theoretical values of the reduced static strength by equation (15)

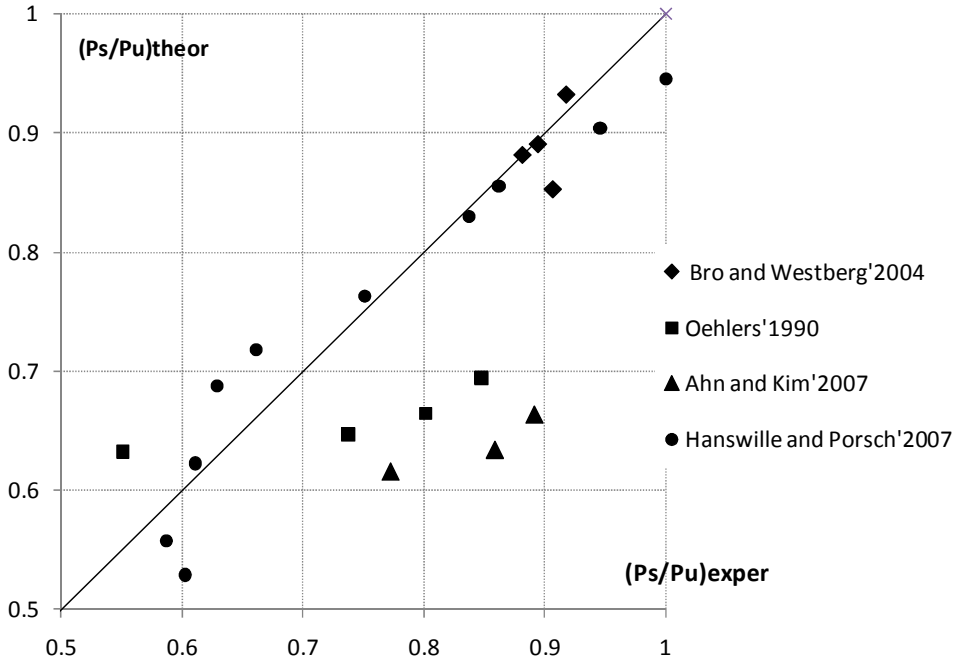


Figure 49 – Comparison of the experimental and theoretical values of the reduced static strength by equation (16)

5.3. Low cycle fatigue behavior of studs

Although most research has focused on loading cases in which shear connectors deform within their elastic range, in recent years, studies have also consider cases where connectors are loaded into their inelastic range. These studies show that loading shear connectors into their inelastic range results in a low number of cycles to reach failure; a phenomenon called low-cycle fatigue.
Reversal of load was rarely considered and full load reversal was considered in specimens tested by Oehlers (1995) and in tests made by Erlicher et al. (2001). Values of load reversal were always well below the stud capacity. Fatigue life was generally in the thousands to millions of cycles and it was determined that shear studs subjected to fatigue loading can have significantly reduced load capacity. From these tests, fatigue-strength relationships were developed. Data was extrapolated into the low-cycle range, inferring that only a slight loss of strength would be expected for a low number of inelastic cycles.
Low-cycle fatigue generally concerns loading approaching or exceeding the yield capacity of a section, with failures occurring prior to 1000 load cycles. Unidirectional testing of shear studs considering low-cycle fatigue was performed by Gattesco and Giuriani (1996) and Gattesco et al. (1997). Fatigue life was shown to be greatly reduced when large slip was present. Testing was performed using a direct shear test method to model low slab compressive stresses at the end of a typical beam section. These researchers, as well as Taplin and Grundy (1997) and Valente (2007), noted the accumulation of slip and damage under reversed cyclic loading. Shake down (stabilization of residual deformations in a structure during a series of repeated loading beyond the elastic limit) did not occur, but instead incremental collapse (increasing residual deformations until failure) was verified.

5.4. Slip characteristics for studs under repeated loading

Hallam (1976) found that the slip per cycle could be related to the load range by equation (17), where, the slip value is expressed in millimeters.

$$\log \delta = -10 + 12.99\left(\frac{\Delta P}{P_u}\right) \tag{17}$$

Like Hallam, Lo (1978) found that the slip increases approximately linearly with cycles of loading. He did not develop a relationship between the rate of slip growth and the magnitude of the repeated load.
Meanwhile, Oehlers and Foley (1985) found the same principle. By assuming that the stiffness reduction is attributable to the propagation of a fatigue crack, they proposed that the equation for the rate of slip growth as a function of the load range should be similar in form to Paris's equation, that is

$$(\frac{\delta}{d})\alpha(\frac{\Delta P}{P_u})^m \tag{18}$$

By reviewing the data from Hallam and Lo, as well as their own tests results, the authors determined the value for the coefficient m as given in Table 30.

Table 30 -Coeddicient in equation (18) for the rate of slip growth as determined by Oehlers and Foley (1985)

Data source	*m*
Hallam (1976)	6.7
Lo (1978)	5.0
Oehlers and Foley (1985)	4.0
All above	5.3

In the following, Oehlers and Coughlan (1986) combined the results from above authors and their own data and proposed equation (13) for the rate of slip growth as:

$$\frac{\delta}{d} = 1.70 \times 10^{-5}(\frac{\Delta P}{P_u}) \tag{19}$$

Later, Taplin (1997) proposed equation (20), which corresponds to the line of best fit on the date presented in Table 26 and Table 27. The value of slip per cycle is expressed in millimeter:

$$\delta = 10^{(3.71\frac{P_{\max}}{P_u}-4.91)} \tag{20}$$

Valente (2007) proved that the evaluation on the rate of slip growth is influenced by the number of load cycles performed or considered in an analysis for headed studs embedded in lightweight concrete. The linear and the logarithmic trends were analyzed, since they gave the best correlations between rate of slip growth and number of load cycles applied. The author established linear and logarithmic equations for each specimen. In global terms, the logarithmic trend is a better approach when the load range value is lower: $\Delta P/P_u<0.5$, while the linear trend is best fitting when the load range is higher: $\Delta P/P_u>0.5$.For test CN 13, the linear equation is expressed as:

$$\log\delta = -7.11 + 5.79(\frac{\Delta P}{P_u}) \tag{21}$$

The comparison of this equation and the results obtained by other authors is represented in Figure 50. Although the equation (21) has a similar manner as the equation (18), but the values obtained for the rate of slip growth are always higher than the values obtained by Lo. With exception on the results obtained by Taplin (1997), it seems that the rate of slip growth is higher for lightweight concrete than for normal density concrete.

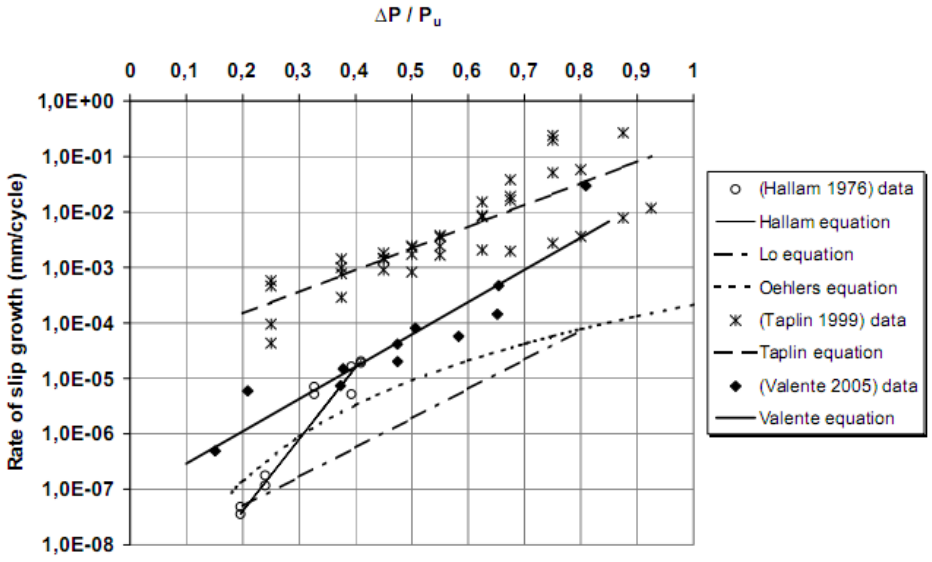


Figure 50 – Evolution of slip depending on the cycles load range by Valente (2007)

Furthermore, based on their slip test results, Hanswille and Porsch (2007) established the load-slip equation for headed studs under cycling loading, the relationship between applying load P_i (kN) and the corresponding slip s_i (mm) under a certain cycling numbers N_i could be expressed as:

$$P_i = 1.4P_u \cdot \left[\begin{array}{l} s_i - 0.104e^{3.95\cdot\frac{P_{\max}}{P_u}} \\ +(0.664\dfrac{P_{\max}-\Delta P}{P_u}+0.029)\cdot\ln(\dfrac{N_f}{N_i}-1) \end{array} \right] \qquad (22)$$

with the limitation for $\Delta P \le 0.8P_u$. It seems to have a good prediction for their experiments. But the general application of this equation needs more research.

6. Conclusion

This report deals with the question of fatigue strength of shear connectors. The test results of four kinds of shear connectors as headed steel stud, channel connector, post-installed shear connector and Perfobond connector are collected, including the test details. The results are divided in three parts: endurance tests, static residual strength after a certain number of high-cycle tests and the slip behavior under fatigue loading. On the basis of test results from these three parts, a series of problems were carefully discussed such as the fatigue life calculation models, the residual strength calculation models, the low-cycle fatigue behavior and the slip characteristics. Based on the research, several conclusions could be derived..

(1) For headed studs, since 1960s, various researches have conducted a great number of fatigue tests on push-out or beam test specimens, but because of different sizes and shapes of the composite components, because of number and position of studs, because of various

restraints, the results from international test vary widely. The factors as loading mode, load range, peak load, weld quality, stud diameter, concrete ability and static strength all seem to have an influence on the fatigue strength of shear studs. For the test of new shear connectors, the standard push-out specimens in Eurocode 4 is recommended.

(2) Earlier researchers view stress range is the main factor influencing high cycle life of studs, and some provisions are established basing on their research. But researches in recent years prove that the peak load and the static strength of studs also have remarkable influences, since the crack of concrete around studs may redistribute the forces and cause more damage. New principles including these effects for calculation of fatigue endurance are established and the future application of these principles need more research.

(3) Headed studs subjected to bidirectional cyclic loading have a higher fatigue strength compared to unidirectional cyclic loading, since fracture always starts at the side of the stud which is subjected to bending tensile stress, it is not the full double amplitude under reversal stress, which is responsible for damage. The test results of large diameter studs like 25 mm, 28 mm, 30 mm and 32 mm show that the fatigue endurance of large diameter is slightly lower that Eurocode 4 predicts. For the shear studs embedded in lightweight aggregate concrete, a somewhat more inclined S-N curve could be observed.

(4) There are linear and nonlinear two kinds of theories for calculation residual strength of headed studs, both have good prediction with experimental results, but the linear model has more restriction. And there are two trends to predict the slip behavior of shear studs too, linear and the logarithmic trends, which applied to describe the relationships between the slips with the maximum load and the ultimate static load.

(5) For channel connector, the limited test results show that the stress range at the weld position of the channel is the main factor for fatigue strength of this kind connector. And it is proved that the fatigue strength of the single channel specimen was equivalent to the fatigue strength of specimens with two channel shear connectors. The orientation of the channel connector whether facing toward or away from the applied load had no significant influence on the fatigue life.

(6) For post-installed shear connectors, the test results show that they have significantly higher fatigue strength than conventional welded shear studs. The superior fatigue strength of these post-installed shear connectors enables the strengthening of existing bridges by using fewer shear connectors. But for their usage in new composite beams, spaces should be set aside in advance of concreting.

(7) Perfobond showed extremely high fatigue strength compared to welded studs, but the influencing factors for their fatigue behavior are not clear yet and should be well researched in the future.

7. References

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