Influence of transverse loading onto push-out tests with deep steel decking

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ABSTRACT

This paper presents the results of 20 push-out tests on shear stud connectors, placed centrally in the ribs of 58 mm and 80 mm deep steel decking. The tests were designed to investigate the realistic load–slip behaviour of the shear connectors and the influence of transverse loading. The tests considered two different stud diameters and the effect of concentric and eccentric transverse loading. In addition, the influence of a second layer of reinforcement, the welding procedure and the number of shear connectors in each rib have been considered. The observed influence of these parameters on the load–slip behaviour is presented and explained with regard to material properties and load-bearing models. In addition, the test results are compared with the current analytical approaches, which are shown to be non-conservative in some cases, because the presented deck shapes were not well considered in the development and calibration of EN 1994-1-1.

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1. Introduction

The application of composite beams and slabs has many advantages in terms of economic construction of multi-storey buildings due to the increase of stiffness and load-bearing capacity of the structure. Most commonly, composite action and transfer of shear forces between the steel beam and the slab is ensured by use of headed shear studs that are welded to the top flange of the beams. The current rules in EN 1994-1-1 for the analysis of the shear connector resistances are based on the failure modes of studs in solid slabs and do not sufficiently consider the load-bearing behaviour of studs in the ribs of slabs with modern deep steel decking. Also, the push-out testing procedure, described in Annex B of EN 1994-1-1, was originally defined for solid slabs. This setup, when applied to slabs with steel decking, leads to lower resistances and deformation capacity of the shear studs in comparison with beam test results. The paper develops an appropriate push-test method and assesses various test parameters, such as deck shape, shear connector size, reinforcement pattern and concentric and eccentric transverse loading, which have not been studied previously.

1.1. Load-bearing behaviour of shear connectors

The load-bearing behaviour of shear studs in solid slabs is shown in Fig. 1. The shear connectors initially transfer the shear force P by a compression force A acting on the concrete. The compression force A pushes

against the weld collar at a shallow angle β. With increasing load, the concrete in front of the stud is damaged and the shear force moves to a higher position into the stud shank. This leads to plastic bending and shear deformations. Because of the fixed support conditions of the head of the stud, a tension force C develops in the stud shank. The tension force C is in equilibrium with a compression cone in the surrounding concrete. The compression struts in the concrete activate friction forces D between the slab and the steel flange. Finally, failure occurs in the stud shank above the weld collar because of combined tension and shear forces.

When the shear stud is placed in the deck rib of a composite slab, the load-bearing behaviour differs from the behaviour of studs in solid slabs, as shown in Fig. 2. The deck rib geometry has a strong influence onto the load-bearing behaviour. In general, two loading stages can be characterised by the two load peaks P_1 and P_2. The first peak load P_1 is reached when the concrete in front of the stud is crushed and two plastic hinges have developed in the stud shank. At higher slips, the support conditions of the head of the stud lead to a back-anchorage effect. Thus, the head of the stud introduces compression forces into the still intact concrete section which are in equilibrium with the tension force C in the stud shank. This effect allows the development of a second peak load P_2. Finally, failure occurs in form of a concrete pullout cone or stud rupture.

The development of this failure mechanism requires a sufficient embedment depth of the head of the stud into the continuous part of the concrete slab topping. If the embedment depth is relatively small, the support reaction of the head of the stud cannot be introduced into the concrete. In these cases, the concrete fails in a brittle form and a failure

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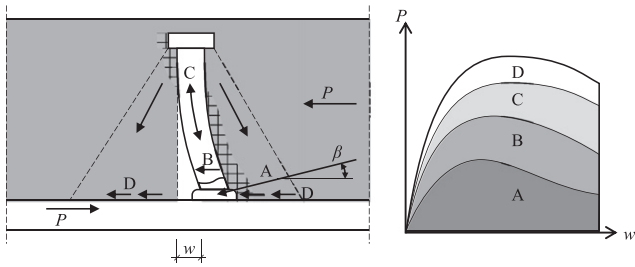 *E-mail address:* christoph.odenbreit@uni.lu (C. Odenbreit).

Fig. 1. Load-bearing behaviour of shear studs in solid slabs according to Lungershausen [1].
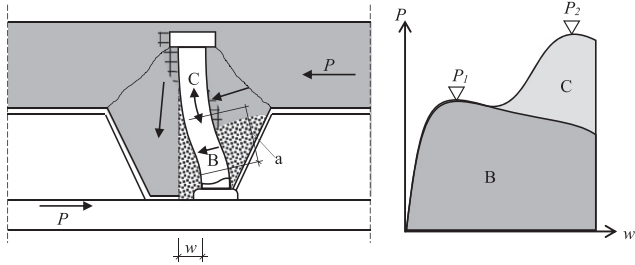

Fig. 2. Load-bearing behaviour of shear studs placed in the ribs of composite slabs according to Lungershausen [1].

mechanism with only one plastic hinge develops (see Fig. 3). Therefore, the behaviour of the shear stud is also influenced by the geometry of the steel decking and the shear stud itself.

In addition to the stresses that are introduced into the concrete directly by the shear stud, additional stresses occur because of the loading of the concrete slab itself. The loading of the slab leads to stresses resulting from vertical loads and bending moments acting on the slab at the line of the shear connectors. These stresses affect the crushing of the concrete in front of the stud, as higher stresses can be reached in multi-dimensional compression. The embedment conditions of the head of the stud may also be influenced by the development of large cracks. These effects are not yet well investigated and so far not considered in the push-out test as proposed in EN 1994-1-1 Annex B [2].

1.2. Test setups to investigate the load–slip behaviour

The push-out test specimen for solid slabs, as given in EN 1994-1-1 Annex B2 [2], is shown in Fig. 4. The distribution of the shear forces according to Roik et al. [3] is suitable to reflect the behaviour in real beams with solid concrete slabs.

However, when deep steel decking is used in concrete slabs, the obtained load–slip behaviour from push-out tests can result in up to 30% lower stud resistances and lower displacement capacities than in composite beam tests using similar configurations [4].

The load–displacement behaviour of a push-out test is strongly dependent on the boundary conditions of the concrete slab. Specimens
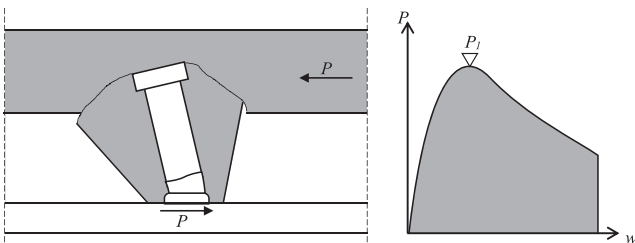

Fig. 3. Failure of ribs because of a too small embedment depth of the stud according to Lungershausen [1].
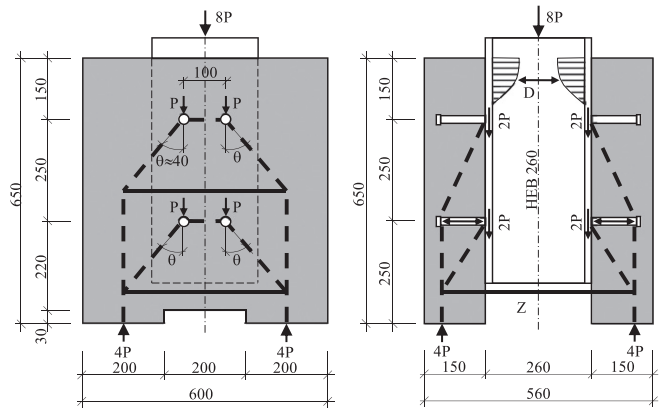

Fig. 4. Dimensions of the push-out test specimen according to EN 1994-1-1 Annex B [2] and force distribution according to Roik et al. [3].
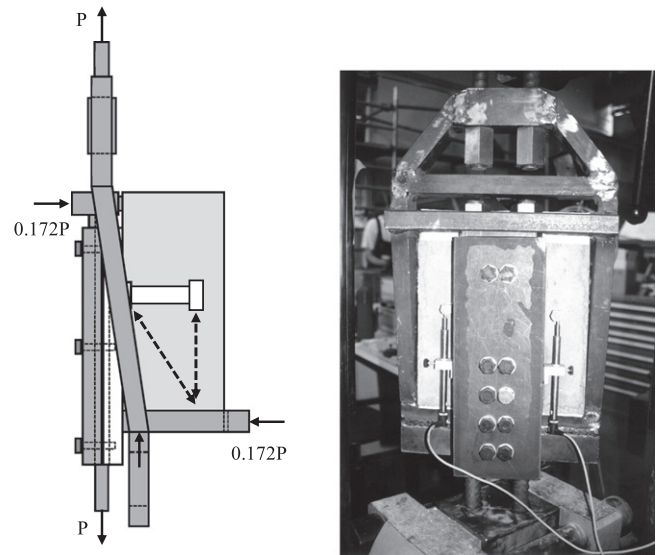

Fig. 5. Single push-out test used by Döinghaus [5].

with sliding bearings may underestimate the real shear resistance, whereas for tests with tension ties or rigid horizontal restraints, the shear resistance may be overestimated [5–7].

The differences of the behaviour of the shear connection in push-out tests and in beam tests led to the development of alternative test setups over recent years, such as the single push-out test [5] (see Fig. 5) and the horizontal push-off test [6,8] (see Fig. 6).

The horizontal push-off test represents a small step towards the consideration of transverse loads because the self-weight of the slab is taken into account. Other research [4,10] explicitly applied transverse loads to normal push-out specimens (see Fig. 7). Typically, concentric loading positions were used. Currently, the degree of transverse loading that should be applied in these tests is under discussion. According to Hicks and Smith [4], who investigated transverse loads of 4% to 16% of the shear load, a value of 12% transverse load was suitable to represent
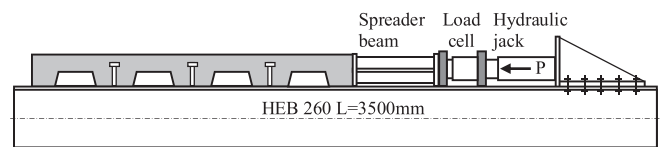

Fig. 6. Horizontal push-off test used by Lam et al. [9].

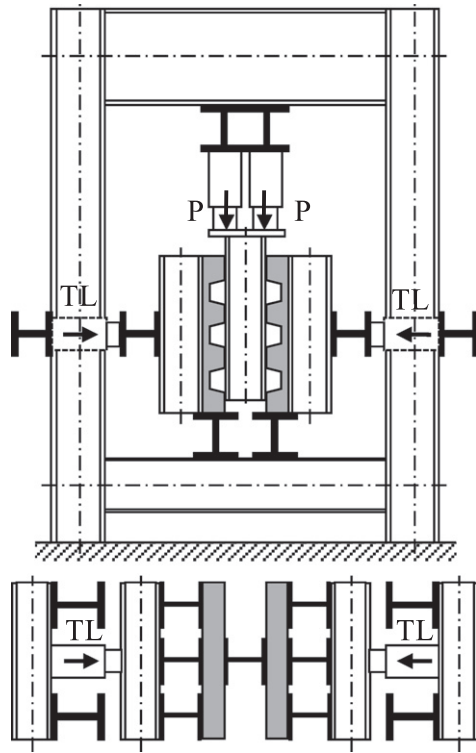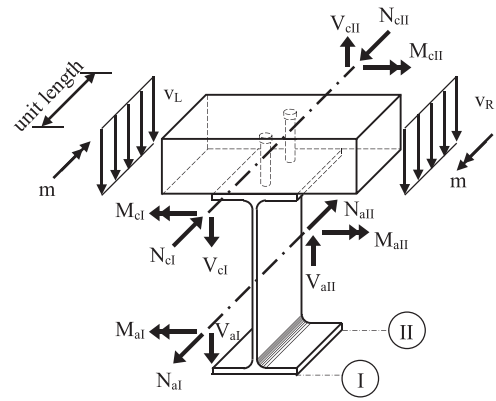(a) Internal forces and moments in a composit beam.

Fig. 7. Transverse loaded push-out test used by Hicks and Smith [4].

the behaviour of the shear studs in the accompanying composite beam tests.

2. Consideration of transverse loading in the presented push-out tests

The experimental results of Hicks et al. [4,10] showed an improvement of the shear stud resistance for slabs with steel decking, when concentric transverse loads were applied. The beneficial influence of transverse loads is not considered in the assessment of the shear resistance of studs according to EN 1994-1-1 [2] or the other design equations presented in this paper [1,11], and the mechanisms of how the transverse load influences the load-bearing behaviour of the shear connection, shown in Figs. 1 to 3, have not been established. In addition, the influence of the negative moment of the slab was not considered. To respect these effects in the presented push-out tests and in a later engineering model of the shear resistance of headed studs [12], transverse loads were applied with an eccentricity, as shown in Fig. 8c.

To investigate the influence of transverse loading on the load-bearing behaviour of headed studs, it is necessary to define the degree of transverse loading for the conduction and evaluation of the tests. Fig. 8a shows the internal forces and moments acting on a composite beam and its concrete slab. The structural analysis of the slab results in the vertical forces v_R and v_L and the negative bending moment m. The vertical forces v_R and v_L lead to compression in the shear interface between the slab and the steel profile. The transverse load, TL, is then the sum of the vertical forces of the slab according to Eq. (1). The structural analysis of the composite beam results in internal forces and moments acting on the slab and the steel section as shown in Fig. 8a. In Section I acts, the concrete compression force N_{cI}, and in Section II acts, the concrete compression force N_{cII}. The difference of these compression forces is the shear force, P, that is transferred between Section I and Section II, as shown in Eq. (2). The degree of transverse loading, ρ, is then the ratio of the transverse load, TL, to the shear force, P, as shown
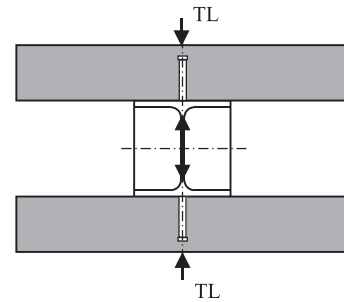
(b) Concentric transverse loading of pus-out tests.
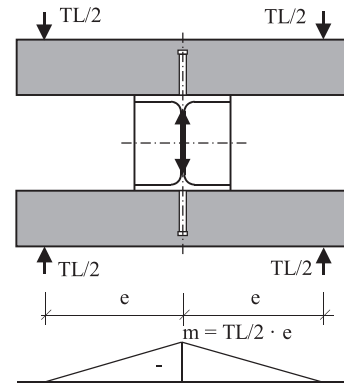
(c) Eccentric transverse loading of pus-out tests.

Fig. 8. Internal forces and moments at the line of the shear studs in a composite beam and their reflection in a push-out test.

in Eq. (3):

$$TL = |v_L| + |v_R| \tag{1}$$

$$P = N_{cII} - N_{cI} \tag{2}$$

$$\rho = \frac{TL}{P} \tag{3}$$

where

TL   transverse load acting on the shear interface

v_L, v_R vertical forces of the slab
N_{cI} concrete compression force in Section I
N_{cII} concrete compression force in Section II
P shear force between steel beam and concrete slab between Section I and Section II
ρ degree of transverse loading

A preliminary study was conducted to define reasonable values for the degree of transverse loading. A single-span composite beam as inner support of a two span composite slab with an imposed load of $q_k = 3.5$ kN/m^2 was considered (see Fig. 9). Nominal material properties of steel grade S 355 and concrete grade C 30/37 were assumed. Two different deck shapes with a deck height of 58 and 80 mm were considered. The shear stud resistance was calculated according to EN 1994-1-1 [2].

The results of this study are shown in Fig. 10. The degree of transverse loading, ρ, is mostly dependent on the type and span of the decking and the type of shear connector. Deeper decking led to higher degrees of transverse loading because the shear force P per stud was smaller than for shallow decking. Longer beam spans led to lower degrees of transverse loading because the bending moment, and hence the required shear force, increased.

For typical spans of slabs with 58 mm deep decking, degrees of transverse loading between 6.6% and 12.2% were obtained, while for 80 mm deep decking, the values were between 13.0% and 20.8%. Thus, the transverse load of 12% proposed by Hicks and Smith [4] could not be achieved in all cases considering the loading conditions of the composite beam.

The transverse loads in the presented test programme were chosen to reflect degrees of transverse loading of 8% and 16%, which reflect typical values for the two considered deck shapes. To investigate the effect of the eccentricity, e, the push-out tests have been conducted with concentric transverse load application ($e = 0$, see Fig. 8b) and eccentric transverse load application (in which $e = 380$ mm, see Fig. 8c).

3. Test programme and setup

3.1. Test setup for transverse loading

A possible test setup for the application of transverse loads was developed by Hicks [4], as shown in Fig. 7. This setup has the benefit that
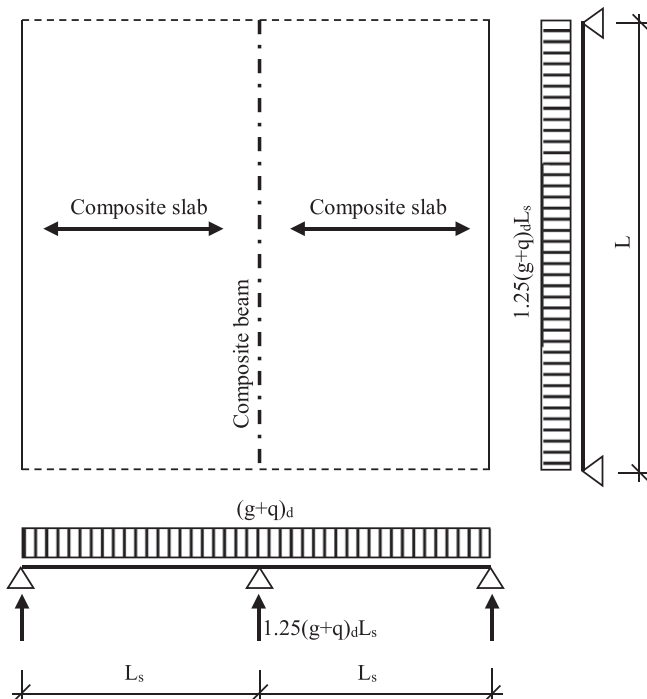
for each shear interface there was a horizontal hydraulic jack to apply the transverse load. For the tests presented in this paper, only one hydraulic jack was used for the application of the transverse load. Therefore, a clamping device as shown in Figs. 11 and 12 was used.

The vertical shear load is applied directly to the specimen by the primary hydraulic jack, as shown in Fig. 11a. Using the clamping device, the horizontal secondary jack applies the transverse load to both slabs of the test specimen. The secondary jack pushes with the force TL on the elements 'HP2' and 'HP3' to apply a concentric transverse load to slab 'S1', see Fig. 11b. At the same time, the secondary jack pushes with the force TL on element 'HP1'. Drawbars are used to transfer the force TL from element 'HP1' to 'HP5', see Fig. 11b. This means that the secondary jack pulls the elements 'HP5' and 'HP4' to apply the concentric transverse force TL to slab 'S2'.

To ensure that only the influence of the transverse loading is investigated and for eccentric loading the bending of the slabs is not restraint, the friction at the supports of the specimen is minimised with pads of polytetrafluoroethylene (PTFE).

Some tests were conducted without transverse loading for comparability to other data sources. In these tests, the slabs were placed on a mortar bed. A tension tie, as shown in Fig. 1, was not applied.

The vertical hydraulic jack, which applied the shear force to the specimen, was displacement controlled when loading the specimen to failure. The horizontal hydraulic jack, which applied the transverse load, was force controlled. Two different methods were used to adjust the applied transverse load during the test:

1. 'Variable transverse load'
   For the first test of a series, the load-bearing capacity of the specimen is unknown. To ensure that the applied transverse load confirms to the desired degree of transverse loading, ρ, the procedure of a variable transverse load is used. In this case, the vertical test load, $2P$, is continuously measured. The transverse load, TL, applied by the horizontal hydraulic jack is controlled by a programme that calculates continuously the required transverse load from the currently measured vertical test load: TL $= \rho \cdot P$. During the test, the control programme continuously adjusts the force applied by the horizontal hydraulic jack according to the calculated value of the transverse load.

2. 'Constant transverse load'
   When the load-bearing capacity of the specimen can be estimated, for example, from the results of a test with variable transverse load, the procedure of a constant transverse load can be used. Based on the estimated load-bearing capacity and the desired degree of transverse loading, the force that must be applied by the horizontal jack is calculated. The transverse load is applied by the horizontal hydraulic
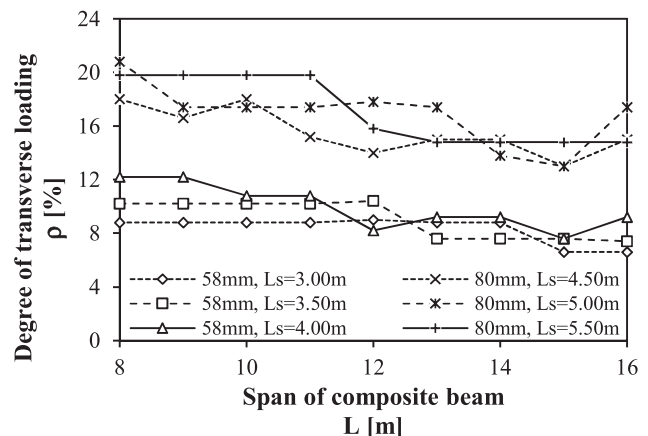
Fig. 9. Static system used for parametric studies on the degree of transverse loading.

Fig. 10. Degree of transverse loading, ρ, for different deck shapes.

(a) Elevation

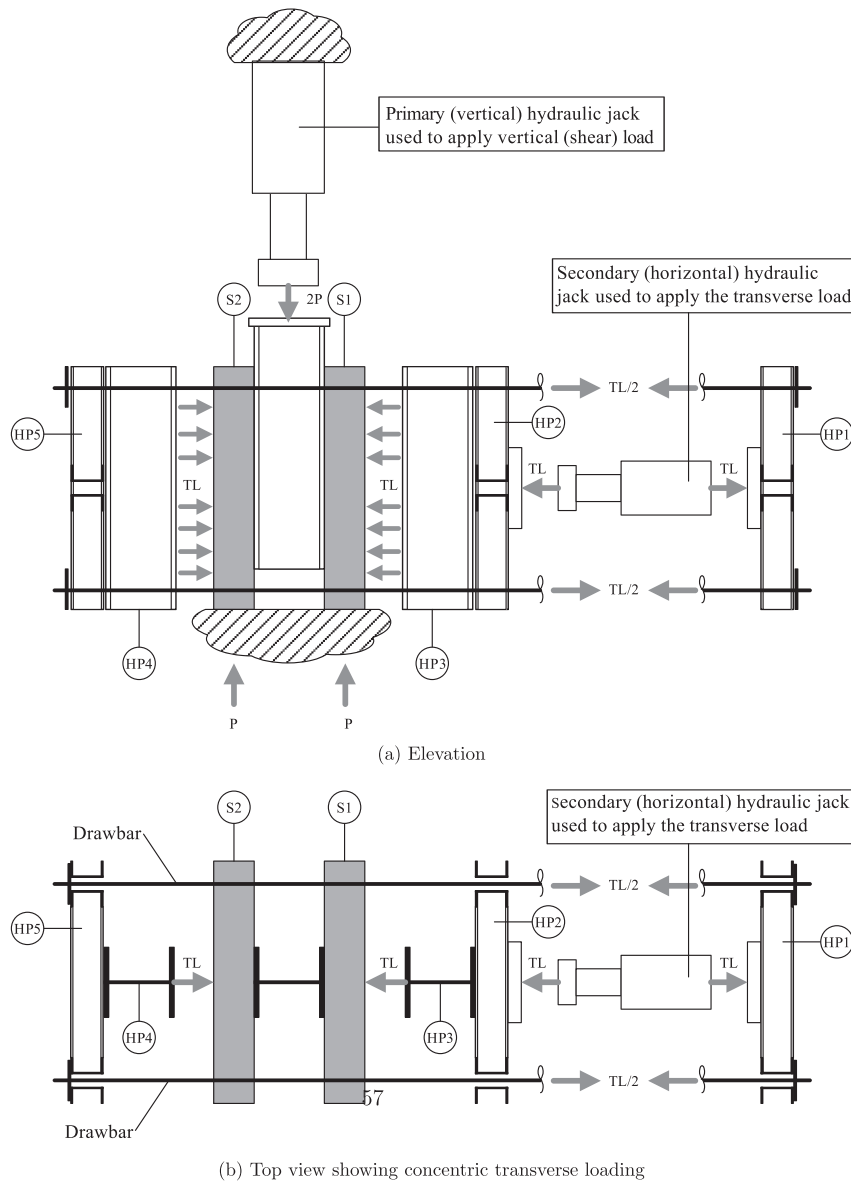(b) Top view showing concentric transverse loading

Fig. 11. Schematic view of the clamping device for the application of transverse loads.

jack before application of the vertical load. During the test, the horizontal jack is set to maintain a constant force: TL = constant.

3.2. Test programme and material properties

An overview of the test programme is shown in Table 1 and Figs. 13 and 14 show the dimensions of the test specimens.

The tests in series 1-04 to 1-06 investigated the use of 58 mm deep decking, in which the concrete strength was about 30 N/mm². Two layers of reinforcement were placed in the slabs, and the optional recess according to EN 1994-1-1 Annex B [2] at the bottom of the slabs, as shown in Fig. 13, was used. A reinforcement bar of 20 mm diameter was placed above the recess to prevent vertical splitting of the specimen. The steel decking was pre-punched and single shear studs per rib with a diameter of 22 mm were welded directly to the flange of the beam. The stud height after welding was measured at about 124 mm, which results in an embedment depth of about 3.5 diameters into the concrete above the decking. Thus, the required minimum embedment of 2 diameters, according to EN 1994-1-1 [2], was satisfied.

Series 1-04 and 1-05 investigated the influence of concentric transverse loads. According to the results of the study presented in Section 2, the slabs had a transverse load of about 4% of the test load in series 1-04 and about 8% of the test load in series 1-05. Thus, the degree of transverse loading at each shear interface was about 8% and 16%.

Series 1-06 investigated the influence of the negative bending moment in the slab at a transverse load of about 4% of the test load. The degree of transverse loading at the shear interface reflects the typical value of about 8%, which was found in the study presented in Section 2. The transverse load was applied with an eccentricity of 380 mm.

The tests in series 1-09 to NR1 used 80 mm deep decking. These tests used a single layer of reinforcement, except for series 3-01, which had two layers of reinforcement. Pairs of shear connectors at a transverse spacing of 100 mm were welded through the decking in series 1-09 to 3-01. Series NR1 had single studs per rib that were welded through the decking and a single layer of reinforcement was placed in the slab. The shear connectors were 19 mm diameter, and the height after welding was about 119 mm for through deck welded studs and about 121 mm for studs welded directly to the beam. Thus, the embedment depth of the head of the stud into the concrete above the decking was
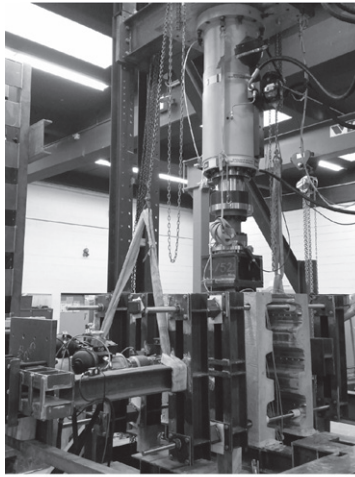
(a) Parts HP1, HP2 and HP3.

(b) Parts HP4 and HP5.
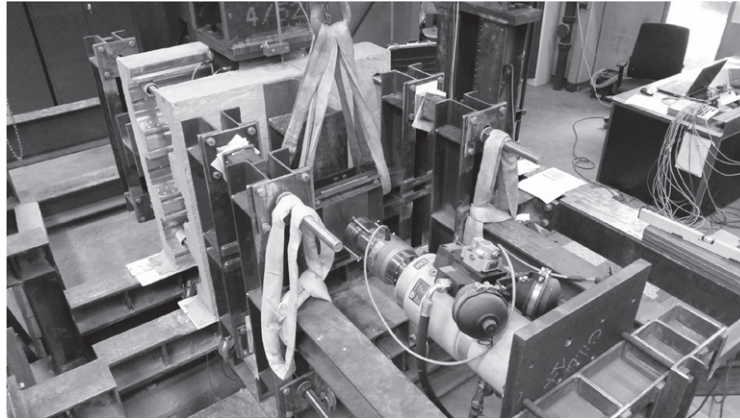
(c) Hydraulic Jack used for the transverse force application

Fig. 12. Test setup for the application of transverse load assembled for concentric loading.

2 to 2.2 diameters. All specimens with 80 mm deep decking were conducted without the recess at the bottom of the slab, as shown in Fig. 14.

The tests in series 1-09, 1-10 and NR1 investigated the influence of concentric transverse loading, while the influence of the eccentricity was investigated in series 1-11.

For all specimens, the concrete was cast in a horizontal position to reflect the real conditions. The two halves of the specimen were welded together prior to testing. The cylinder strength and Young's modulus of the concrete were measured at 28 days and at the day of the test respectively and are shown in Table 1. The tensile strength, f_u, of the shear connectors was measured as 551 N/mm^2.

4. Observed load–slip curves and failure modes

4.1. General results of tests with 58 mm deep decking

Typical load–slip curves for the tests with 58 mm deep decking with concentric and eccentric transverse loads are shown in Fig. 15. All tests exhibited a load-bearing mechanism with two plastic hinges as described by Lungershausen [1]. All tests showed a second load peak, but this peak decreased when eccentric transverse load was applied. Identical specimens without transverse loading, presented in [13], showed a similar behaviour with two load peaks. The observed failure modes were:

- Rib punch-through, see Fig. 16
- Concrete pullout, see Fig. 18
- Stud failure, see Figs. 17 and 21a

4.2. General results of tests with 80 mm deep decking

Examples of typical load–slip curves for specimens with 80 mm deep decking are shown in Fig. 19. The observed failure modes were

- Rib pry-out, see Fig. 20
- Stud failure, see Fig. 21b

Fig. 19 shows that there was no significant difference between tests with single studs per rib (NR1-1) and pairs of shear connectors (1-10-3) within the first 20 mm of slip. The load–slip curves were linear until a brittle failure of the concrete ribs occurred. This led to an immediate loss of stiffness and a drop-off in the test load by up to 15%. The static resistance of the tests $P_{e,static}$ was determined at failure. Typically, the studs developed only a plastic hinge at the bottom of the stud shank (see Fig. 20c), except for tests with high transverse compression. The behaviour after the failure of the ribs was strongly dependent on the transverse loading, as shown by the comparison of specimens 1-10-1 and 1-10-3, shown in Fig. 19. In addition, the boundary conditions of

Table 1
Specimen geometry, concrete strength and loading conditions of push-out tests.

| Series	No.	Decking			Studs			Concrete properties		Reinforcement		Transverse Load	
		h_p	b_m	t	d	h_{sc}	n_r	f_c	E_c			$\sum V$	e
		[mm]	[mm]	[mm]	[mm]	[mm]	[−]	[N/mm²]	[N/mm²]	Bottom	Top	[kN/slab]	[mm]
1-04	1	58	81.5	0.88	22.2	124.3	1	30.6	20,900	Q188A	Q335A	4.1%+	0
	2				22.2	124.0		30.9	21,500			12.5	0
	3				22.2	124.0		30.9	21,500			12.5	0
1-05	1	58	81.5	0.88	22.2	124.0	1	30.7	22,100	Q188A	Q335A	8.2%+	0
	2				22.2	123.8		30.7	22,100			25.0	0
	3				22.2	123.9		32.6	22,800			25.0	0
1-06	1	58	81.5	0.88	22.2	123.6	1	29.9	21,200	Q188A	Q335A	4.1%+	380
	2				22.2	124.1		31.1	21,400			12.5	380
1-09	1	80	137.5	0.90	19.1	118.8	2*	42.6	28,000	Q188A	–	8.8	0
1-10	1	80	137.5	0.90	19.1	118.6	2*	42.6	28,000	Q188A	–	17.5	0
1-10	2	80	137.5	0.90	19.1	118.1	2*	42.6	28,000	Q188A	–	13.2	0
1-10	3	80	137.5	0.90	19.1	118.2	2*	42.6	28,000	Q188A	–	–	–
1-11	1	80	137.5	0.90	19.1	119.4	2*	42.6	28,000	Q188A	–	3.8%+	380
	2				19.1	118.7	2*	42.6	28,000			17.5	380
	3				19.1	118.6	2*	42.6	28,000			17.5	380
3-01	3	80	137.5	0.90	19.1	118.3	2*	40.4	26,800	Q188A	Q335A	–	–
3-02	1	80	137.5	0.90	19.1	123.4	2	42.6	28,000	Q188A	–	–	–
NR1	1	80	137.5	0.90	19.1	121.3	1*	44.1	25,600	Q188A	–	–	–
NR1	2	80	137.5	0.90	19.1	121.2	1*	45.7	25,600	Q188A	–	8.8	0
NR1	3	80	137.5	0.90	19.1	121.0	1*	44.7	25,600	Q188A	–	17.5	0

h_p: height of the deck profile.
b_m: width of the decking at $0.5h_p$.
t: deck thickness.
d: measured stud diameter.
h_{sc}: measured stud height
n_r: number of studs per deck rib
f_c: concrete cylinder strength
E_c: measured Young's modulus of concrete
$\sum V$: applied transverse load.
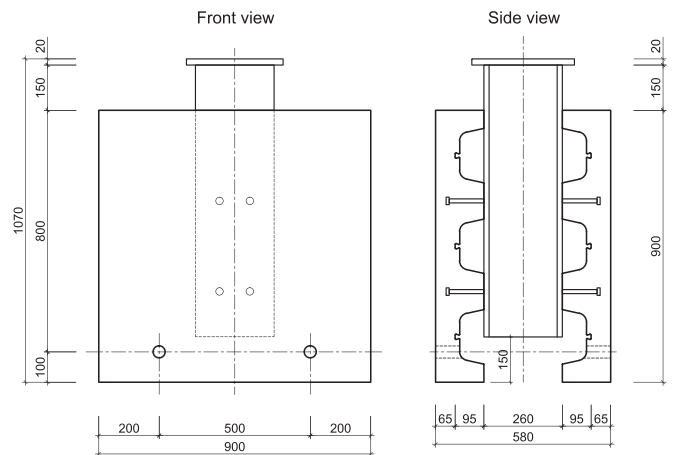e: eccentricity of transverse load
*Welded through the decking
+Percentage of the total test load, permanently adjusted
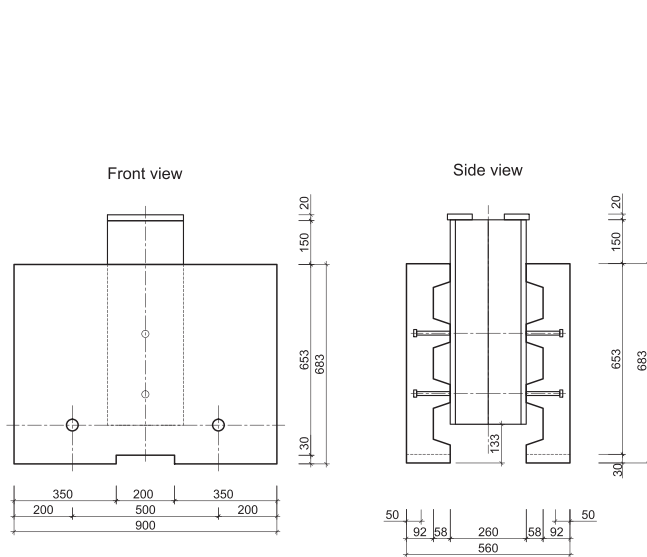
(a) Specimen dimensions
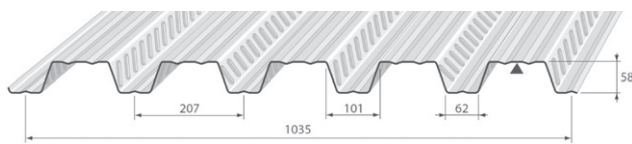
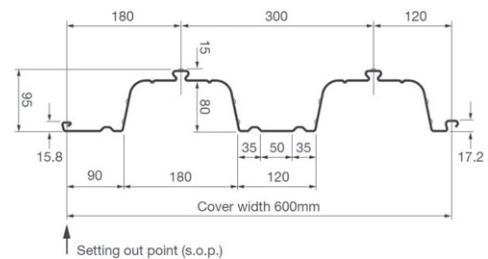
(b) Dimensions of 58 mm deep decking.

Fig. 13. Typical dimensions of specimen with 58 mm deep decking.

(a) Specimen dimensions.

(b) Dimensions of 80 mm deep decking.

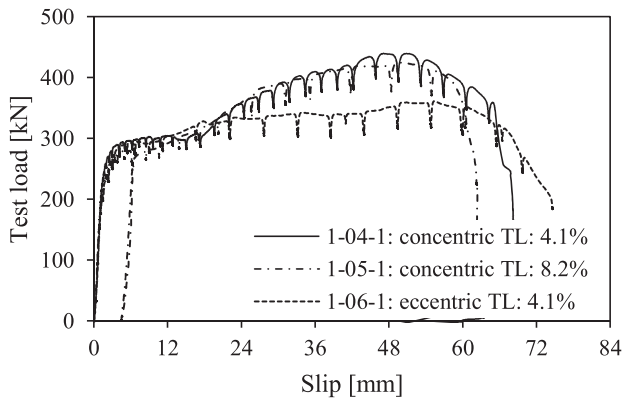**Fig. 14.** Typical dimensions of specimen with 80 mm deep decking.

Fig. 15. Typical load–slip curves for push-out tests with 58 mm deep decking with concentric transverse loading (TL) and eccentric transverse loading (TL).

the test had a significant influence on the load–slip behaviour as reported in [7]. For specimens NR1-1 and NR1-3, failure of the shear studs was observed at about 17 mm slip. The studs failed in the weld collar as there was high porosity in the weld (see Fig. 21b). For specimens with pairs of shear connectors, stud failure was not observed.

Load–slip curves similar to the tests with 80 mm deep steel decking were obtained by Hicks and Smith [4], where shear connectors with a nominal height of 100 mm were welded in the ribs of 61 mm deep decking. In these tests, the studs did not satisfy the minimum embedment depth of 2 diameters, which is required by EN 1994-1-1 [2]. Thus, the support reaction of the head of the stud, which would be necessary for the development of the upper plastic hinge, could not be introduced into the concrete slab and a failure mode with only one plastic hinge developed (see Fig. 20c).

Hicks and Smith [4] referred to the failure of the ribs as concrete pullout. Because of the low slip at failure, the force that would have been necessary to fail the rib of the slab in tension could not have developed in the stud shank. In tests with 58 mm deep decking, concrete pullout was observed at very large slips and appeared as a slowly progressing and ductile failure with two plastic hinges (see Figs. 15, 17 and 18). Failure of the ribs in tests with 80 mm deep decking occurred at a very small slip. Hence, the loading situation at failure was different, and bending

Fig. 17. Stud deformation after concrete pullout.

and shear must have been the dominant loads – instead of tension – leading to different stud deformations and load–slip curves. Therefore, this failure is not treated as concrete pullout but referred to as rib pryout, as shown in Fig. 20.

5. Evaluation of the test results according to EN 1994-1-1 Annex B2

EN 1994-1-1 Annex B2 [2] gives a simplified procedure to determine the characteristic resistance P_{Rk} and slip capacity δ_{uk} out of the results of push-out test. The pre-condition for the application of this procedure is to have a series of three tests with identical nominal properties. The shear stud resistance of each test must not deviate by >10% from the mean value of the shear stud resistance for the series.

Because the tests with 80 mm deep decking varied the degree of transverse loading between 0% and about 10% within some series, they may not be assumed to have identical nominal properties. For information, Table 2 shows the results of the evaluation according to EN 1994-1-1 Annex B2, even if there are <3 tests with identical nominal material properties or loading conditions.

Fig. 16. Rib punch-through failure.

Fig. 18. Concrete pullout failure for the bottom rib and rib punch-through and stud failure for the top rib.

Table 2
Characteristic resistance P_{Rk} and slip capacity δ_{uk} of push-out tests according to EN 1994-1-1 Annex B2.

Series	i	P_e [kN]	$P_{e,static}$ [kN]	P_{Rk} [kN]	δ_u [mm]	δ_{uk} [mm]
1-04	1	295.8	281.8	234.1	67.6	55.3
	2	294.4	278.5		61.5	
	3	274.6	260.1		68.7	
1-05	1	279.8	265.6	239.0	62.1	51.6
	2	316.6	278.5		61.6	
	3	297.7	283.4		57.4	
1-06	1	292.4	275.6	236.5	71.0	59.4
	2	280.5	262.8		66.0	
1-09	1	338.2	312.9	281.6	7.6	6.8
1-10	1	365.1	326.0	293.4	25.7	23.2
1-10	2	371.5	314.3	282.9	5.1	4.6
1-10	3	283.7	249.7	224.8	5.0	4.5
1-11	1	437.1	411.3	29.1	16.9	15.2
	2	421.4	368.3		20.5	
	3	394.6	365.7		22.3	
3-01	3	422.2	378.5	340.7	2.4	2.2
3-02	1	296.0	279.2	251.3	1.8	1.6
NR1	1	316.2	287.4	258.6	3.7	3.3
NR1	2	300.0	271.7	244.5	5.9	5.3
NR1	3	281.8	259.5	233.5	3.3	2.9

P_e: experimental resistance
$P_{e,static}$: static resistance.
P_{Rk}: characteristic resistance for the series
δ_u: displacement capacity
δ_{uk}: characteristic displacement capacity

The characteristic resistance P_{Rk} of a test series is the minimum value of all three tests reduced by 10%. The characteristic resistance P_{Rk}, shown in Table 2, was derived from the static resistances $P_{e,static}$.

For tests with 58 mm deep decking, the characteristic resistance of the specimen was between 234 and 239 kN, which led to a resistance per shear stud of 58.5 to 59.8 kN.

Tests with 80 mm deep decking and pairs of studs showed a characteristic resistances of 225 to 341 kN—i.e. 28.1 to 42.6 kN/stud. The large scatter was related to the brittle type of concrete failure. The resistance improved for higher transverse loads and a second layer of reinforcement. For single studs, the characteristic resistance was between 233 and 258 kN -i.e. 58.4 to 64.7 kN.

According to EN 1994-1-1 Annex B2 [2], the displacement capacity δ_u of a shear stud is the slip at which the test load drops for the first time to the characteristic resistance P_{Rk}. The characteristic slip capacity δ_{uk} is the lowest value of δ_u obtained for the test series reduced by 10%, as shown in Fig. 22. In the presented evaluation, the influence of relaxation is not considered for the determination of the displacement capacity. The results are shown in Table 2.

All specimens with 58 mm deep decking showed a very ductile behaviour with characteristic slip-capacities δ_{uk} between 51 and 59 mm. The slip capacity appeared to be largely influenced by the degree and position of the transverse load.

For 80 mm deep decking, the characteristic slip-capacities were smaller. In most cases they did not satisfy the 6 mm criterion of EN 1994-1-1 [2] to be classified as ductile. However, the general shape of the load–slip curves (see Fig. 19) should allow the assumption of a ductile behaviour in most cases. The problem with the defined slip-capacities arose as the brittle concrete failure led to a relatively large but localised drop-off in the test load that is related to a change of the load-bearing behaviour. The application of further displacement typically led to a slow and ductile decrease of the load.

6. Discussion of influencing parameters

6.1. Influence of variable versus constant transverse loading

For the first tests in series 1-04 to 1-06 and series 1-11, the applied transverse load was continuously measured and re-adjusted to
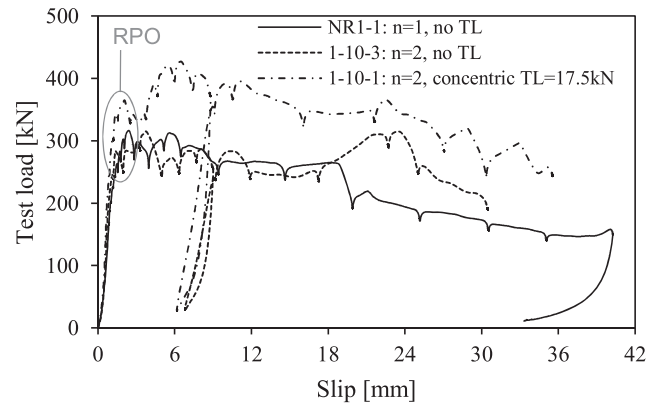
Fig. 19. Typical load–slip curves for push-out tests with 80 mm deep decking, single studs ($n = 1$) and pairs of studs ($n = 2$) without transverse loading (no TL) and concentric transverse loading (TL).

maintain the percentage of the test load given in Table 1. In all other tests, the transverse load was applied with constant values as shown in Table 1. For example, the load–slip curves of series 1-04 and 1-11 are shown in Fig. 23.

Comparing the load–slip curves of tests with variable transverse loads with those of tests with constant transverse loads, no significant influence of the loading procedure on the initial stiffness of the shear connectors and the failure load was observed.

6.2. Considerations on the multi-axial stress state for 58 mm deep decking

Examples of load–slip curves for 58 mm deep decking with different degrees of concentric transverse loading are shown in Fig. 24. The results of series 1-03 [13] and 1-05 are compared with series 1-04 in Table 3. The comparison of series 1-04 and 1-05 shows that higher transverse loads led to an increase of the test load P_e of 3% and a decrease of the displacement capacity δ_u of 8%. The tests reported by Eggert at al. [13], which had an approximately 12 N/mm^2 higher concrete strength, showed an increase of the test load of 26% and a decrease of the displacement capacity of 33%.

The similarity of the effect of transverse load and concrete strength can be explained with the multi-dimensional compression stress state, which increases the failure stress of the concrete [14] (see Fig. 25).

To investigate the influence of multi-dimensional stress states, the concrete strength according to the failure curve of Kupfer et al. [14] was considered in the determination of the analytical resistance for concrete failure according to EN 1994-1-1 [2]. The concrete strength f_c was replaced with the increased concrete strength σ_1, as shown in Eq. (4). The stress σ_2, which was used to determine σ_1, was assumed as the compression stress at the shear interface out of the transverse load

Table 3
Comparison of test results with 58 mm deep decking and different degrees of concentric transverse loading.

		1-03 [13]	1-04	1-05
$\sum V$	[kN/slab]	0	12.5	25.0
f_c	[N/mm^2]	42.5	30.8[a]	32.3
$\frac{f_c}{f_{c,ref}}$	[−]	1.38	1.00	1.02
P_e	[kN]	363.4	288.3[b]	298.0
$\frac{P_e}{P_{e,ref}}$	[−]	1.26	1.00	1.03
δ_u	[mm]	44.3	65.9[b]	60.4
$\frac{\delta_u}{\delta_{u,ref}}$	[−]	0.67	1.00	0.92

[a]Reference concrete strength $f_{c,ref}$
[b]Reference resistance $P_{e,ref}$
[c]Reference displacement capacity $\delta_{u,ref}$

Table 4
Comparison of the analytical resistance for different degrees of transverse loading under consideration of the two-dimensional stress conditions acc. to Kupfer et al. [14].

Series	f_c [N/mm²]	$\frac{\sigma_2}{f_c}$ [−]	$\frac{\sigma_1}{f_c}$ [−]	$P_{Rm,c}$ Eqs. (8), (10), (7) [kN]
1-04[a]	30.8	0	1	529.7
1-05[a]	31.3	0	1	534.4
$\frac{1-05}{1-04}$	1.02	–	–	1.01
1-04[b]	30.8	0.013	1.025	538.4
1-05[b]	31.3	0.025	1.050	552.1
$\frac{1-05}{1-04}$	1.02	1.923	1.024	1.03

σ_1: Increased concrete strength acc. to Kupfer et al. [14].
σ_2: Transverse compression acc. to Eq. (6).
[a] Analysis without consideration of multi-axial stresses.
[b] Analysis with consideration of multi-axial stresses.

(see Eq. (6)):

$$P_c = 0.374 \cdot \alpha \cdot d^2 \sqrt{\sigma_1 \cdot E_c} \qquad (4)$$

$$P_{Rm,c} = k_t P_c \qquad (5)$$

$$\sigma_2 = \frac{\sum V}{n \cdot b_F \cdot b_u} \qquad (6)$$

where

P_c	concrete failure load in solid slab
k_t	reduction factor in EN 1994-1-1
$P_{Rm,c}$	concrete failure with steel decking
$\sum V$	transverse load applied to the slab
$n = 2$	number of deck ribs per shear interface
b_F	width of steel flange
b_u	bottom with of deck rib
$\alpha = 1.0$	for $h_{sc}/d > 4$
$\alpha = 0.2 \cdot (\frac{h_{sc}}{d} + 1)$	for $3 \leq h_{sc}/d \leq 4$

The concrete's Young's modulus E_c in Eq. (8) was assumed to be according to Eq. (7), which was also assumed by Roik et al. [15] in the evaluation of Eq. (4). For the presented analysis, the concrete strength f_c was replaced with σ_1 when Eq. (7) is evaluated:

$$E_c = 9500 \cdot f_c^{1/3} \qquad (7)$$

The comparison was performed with the averaged dimensions and material properties of series 1-04 and 1-05. The resistance was calculated with and without consideration of the two-dimensional failure curve. The obtained analytical resistances $P_{Rm,c}$ are summarised in Table 4.

Without consideration of the two-dimensional stress conditions, the analytical resistance $P_{Rm,c}$ of series 1-05 is about 1% higher than for series 1-04 because of the scatter in the measured material properties and dimensions. The simplified assumption of a two-dimensional stress condition according to Kupfer et al. [14] led to an about 3% higher analytical resistance for series 1-05 compared to series 1-04. In fact, an increase of the averaged measured resistances P_e of about 3% was observed in the tests (see Table 3). Thus, for 58 mm deep decking, the observed influence of concentric transverse loading on the load–slip behaviour is mostly related to the change of the stress conditions in the concrete. The stress conditions can be considered by multi-dimensional material-laws for the concrete. Nevertheless, comparing the test results of Eggert et al. [13] to series 1-04 and 1-05 clearly shows that the effect of a higher concrete strength governs the load–slip behaviour more than the increase of the concrete compressive resistance for multi-axial stress states.

Table 5
Comparison of test results with 80 mm deep decking and different degrees of concentric transverse loading.

Test	$\sum V$ [kN]	P_e [kN/slab]	$\frac{P_e}{P_{e,V=0}}$ [−]	δ_u [mm]	$\frac{\delta_u}{\delta_{u,V=0}}$ [−]
1-10-3	0.0	283.7	1.00	5.0	1.00
1-09-1	8.8	338.2	1.19	7.6	1.51
1-10-2	13.2	371.5	1.31	5.1	1.01
1-10-1	17.5	365.1	1.29	25.7	5.11
NR1-1	0	316.6	1.00	3.7	1.00
NR1-2	8.8	300.0	0.95	5.9	1.59
NR1-3	17.5	281.9	0.89	3.3	0.89

6.3. Influence of the degree of concentric transverse loading for 80 mm deep decking

For the tests with 80 mm deep decking, the effect of the transverse load was more important than for the shallower decking. Especially for tests with pairs of studs, the transverse load strongly improved the load–displacement behaviour (see Fig. 26). This was valid for the failure load of the ribs, as well as for the post-failure behaviour.

For the failure load, an increase of about 30% was observed due to transverse loading (see Table 5). The dependency between the failure load and the applied transverse loads appeared to be linear until a transverse load of about 13 kN was exceeded. For higher transverse loads, the failure load did not increase further (see Fig. 28).

The increase of the failure load and post-failure behaviour was not observed in tests with only one shear stud per rib (see Table 5 and Fig. 27). For single studs per rib, the failure load of the ribs decreased approximately linearly with the transverse load by up to 11% (see Fig. 28 and Table 5). Comparing the load–slip curve without transverse load to the load–slip curve with a small transverse load of 8.8 kN, no significant difference of the behaviour was observed (see Fig. 27). For a higher transverse load of 17.5 kN, the test load after rib pry-out failure was about 50 kN lower than without transverse load until stud failure was observed.

The diversity of the post-failure behaviour for pairs of studs and single studs per rib can be explained with the load-bearing model shown in Fig. 29. Failure is initiated along the surface A-B-C at point A at a slip of about 1 to 2.5 mm. For further loading, the majority of the load must be introduced into the slab along the surface B–C. Due to the inclination of this surface, a shear forces T and a normal force N act on the face B–C. The force N pushes the slab upwards and causes bending moments in the slab—as observed by the deformation of the slabs in most push-out tests (see Fig. 36a). The whole rib rotates around the base of the

Table 6
Influence of the eccentricity on the resistance P_e and the displacement capacity δ_u.

Series	Test	h_p [mm]	$\sum V$ [kN]	e [mm]	P_e [kN]	δ_u [mm]
1-04	1	58	4.1%	0	295.8	67.6
	2	58	12.5	0	294.4	61.5
	3	58	12.5	0	274.6	68.7
	avg.	58	12.5	0	288.3	65.9
1-06	1	58	4.1%	380	292.4	71.0
	2	58	12.5	380	280.5	66.0
	avg.	58	12.5	380	286.5	68.5
$\frac{1-06}{1-04}$					0.99	1.04
1-10	1	80	17.5	0	365.1	25.7
1-11	1	80	3.8%	380	437.1	16.9
	2	80	17.5	380	421.4	20.5
	3	80	17.5	380	394.6	22.3
	avg.	80	17.5	380	417.7	19.9
$\frac{1-11}{1-10}$					1.14	0.77

(a) Concrete failure surface, front view

(b) Concrete failure surface,
side view
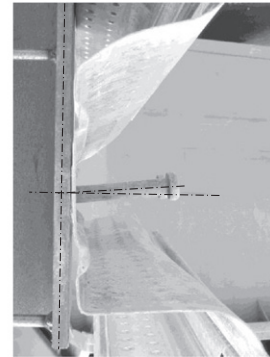
(c) Stud deformation

Fig. 20. Concrete failure surface and stud deformation for rib pry-out failure.

stud and the plastic hinge above the weld collar develops. The head of the stud is supported by the compression strut D. At large slips, plastic bending deformation in the upper stud shank may develop if the bearing capacity of the compression strut D and the face B–C are sufficient. The application of a concentric transverse load restrains the displacement of the slab and increases the force N linearly to the transverse load. For pairs of studs, a higher shear force P was observed with increasing transverse loads. This effect is limited by crushing of the concrete below the surface B–C or the bearing capacity of the compression strut D.

For single studs, the shear force P is not distributed over several studs and consequently the compression strut D is more highly loaded. Also, the surface B–C is smaller than for pairs of studs. Therefore, the concrete below the face B–C crushes at a lower shear force P. The transverse load acting on the slab must be transferred through the rib into the flange of the beam. This changes the compression strut D and the surface B–C in addition, which leads to a decrease of the load-bearing capacity for the shear force P when single studs per rib were used.

6.4. Influence of the eccentricity of transverse loads with 58 mm deep decking

In tests with 58 mm deep decking, no influence of the eccentricity on the first peak load P_e could be observed and the displacement capacity δ_u slightly increased by about 3.9% (see Table 6). Comparisons of the load–slip curves for concentric and eccentric transverse loading showed that the second peak load decreased for the eccentric loaded tests by about 50 kN (see Fig. 30). In addition, the final failure for concentric

(a) Failed shear stud welded directly to the beam.
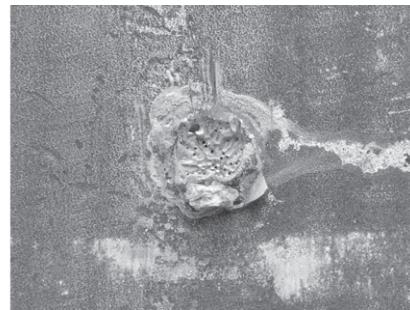
(b) Failed stud welded through the decking.

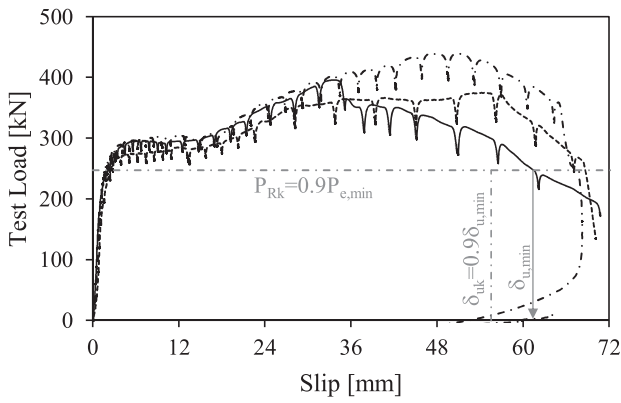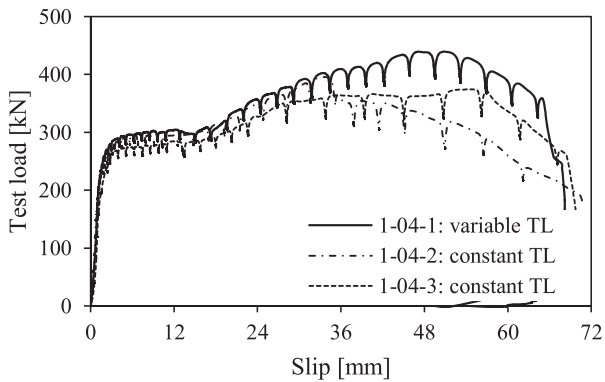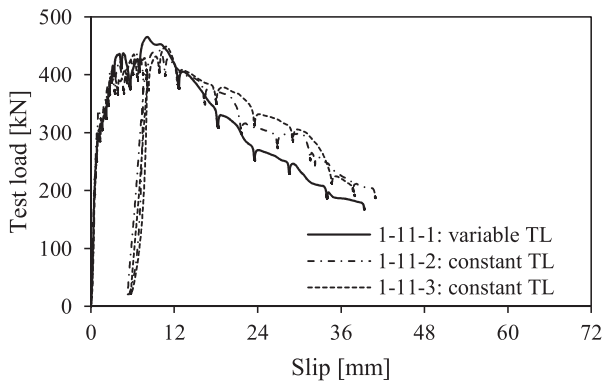**Fig. 21.** Comparison of failure surfaces for shear studs with different welding procedures.

Fig. 22. Determination and δ_{uk} obtained from the load–slip curves according to EN 1994-1-1 Annex B2 [2], shown for series 1-04.

Fig. 24. Comparison of load–slip curves for tests with different degrees of concentric transverse loading for 58 mm deep decking.

transverse loading was stud failure – in most cases – and concrete pull-out. The failure mode observed in the eccentrically loaded tests was always concrete pullout. According to Lungershausen [1], the development of the second peak load is related to the tension force in the stud shank.

The influencing parameters for the observed reduction of the second load peak when eccentric transverse loading was applied are cracking of

the concrete and the larger tension force in the studs. Larger tension forces may occur as the slab slightly lifts at the line of the shear studs because of the bending deformation of the slab. The head of the shear stud restrains this up-lift and so the stud is loaded with an additional tension force.

6.5. Influence of the eccentricity of transverse loads with 80 mm deep decking

Comparing the load–slip curves for concentric and eccentric loaded specimen with 80 mm deep decking (see Fig. 31), no significant influence on the general behaviour could be identified.

All three eccentric loaded specimens showed changes of their stiffness at about 300 to 350 kN, but cracking of the ribs and the drop-off in the test load, which typically occurred after rib pry-out failure, have been observed at test loads of 390 kN to 430 kN. These observations led to the failure loads P_e reported in Table 2. The identified failure loads of eccentric transverse loaded specimens were higher than for the concentric transverse loaded specimen.

Based on the load-bearing model shown in Fig. 29, it can be assumed that the eccentricity has no significant influence on the post-failure

(a) Series 1-04: 58 mm deep decking with concentric transverse load

(b) Series 1-11: 80 mm deep decking with eccentric transverse load

Fig. 23. Comparison of tests with constant transverse load and transverse load maintained relative to the test load.
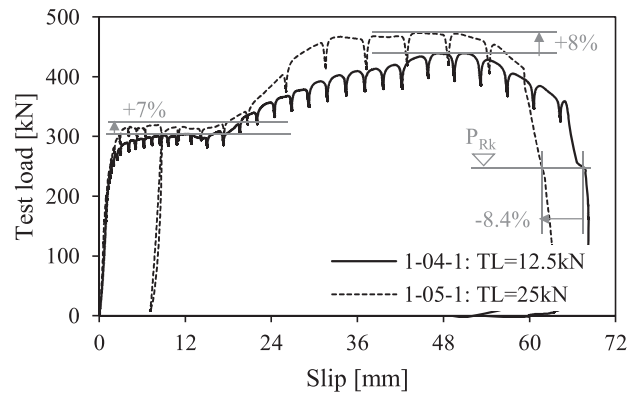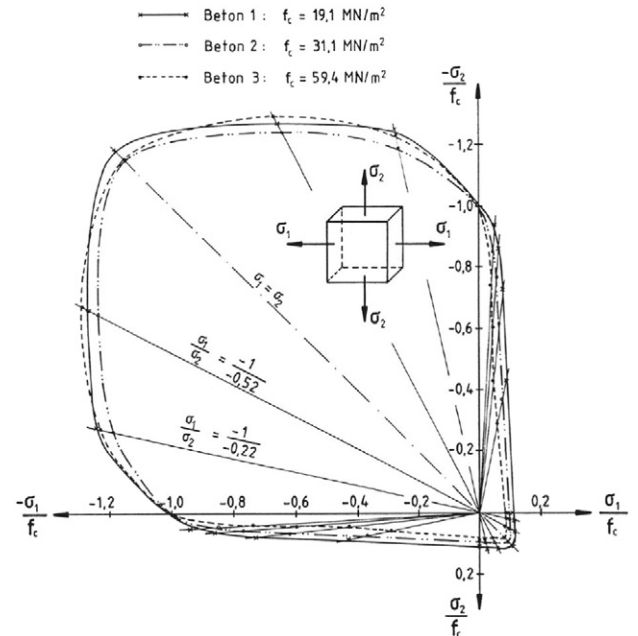

Fig. 25. Failure curves for two-dimensional stress conditions in concrete according to Kupfer et al. [14].
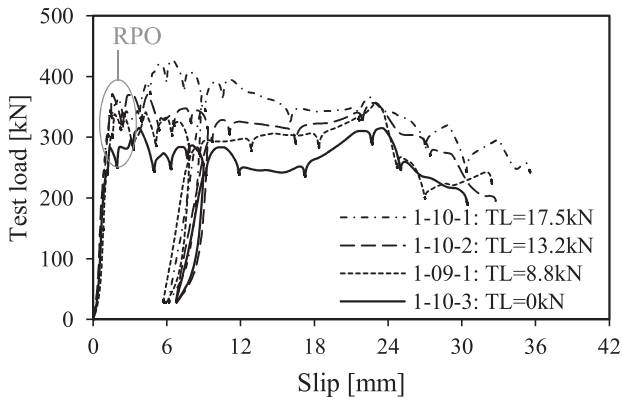
Fig. 26. Comparison of load–slip curves for tests with different degrees of concentric transverse loading and pairs of studs in 80 mm deep decking.

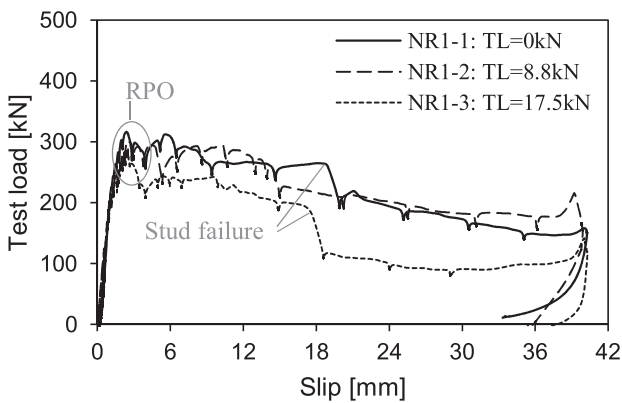Fig. 29. Post-failure behaviour for rip pry-out failure.

Fig. 27. Comparison of load–slip curves for tests with different degrees of concentric transverse loading and single studs per rib in 80 mm deep decking.

Fig. 30. Comparison of load–slip curves for tests with concentric and eccentric transverse loading for Cofraplus 60.

behaviour, because the failure cone is not influenced by cracking of the concrete topping. It is not possible to tension the shear stud due to bending of the slab and the failure surface allows only compression and shear forces to be transferred at the face B–C.

6.6. Influence of through deck welded studs with 80 mm deep decking

Fig. 32 shows shear studs that were welded through the decking and shear studs that were welded directly to the flange of the beam when pre-punched decking was used.
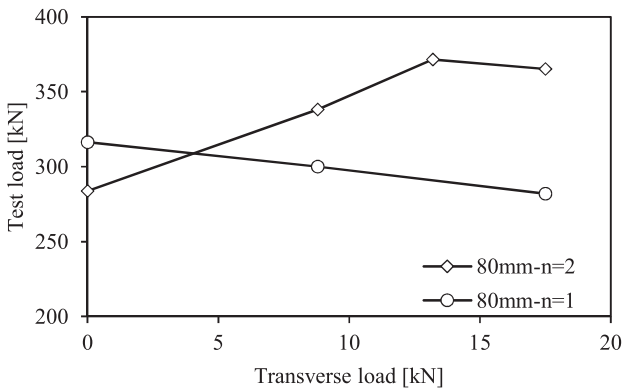
Fig. 33 shows the load–slip curve of a push-out test with shear studs welded through the decking in comparison with a test with a pre-punched decking.

It can be seen that the welding procedure had no influence on the failure load, because rib pry-out failure occurred in both cases at a test load of about 280 kN to 300 kN. The test with the pre-punched decking showed a reduced performance for the post-failure behaviour. The load did not increase again until the steel sheeting came into direct contact with the weld collar. On the other hand, through deck welded studs immediately activate the decking as a tension tie. An additional component for the shear force can be transferred by this tension effect of the

Fig. 28. Test load plotted versus the transverse load for tests with 80 mm deep decking with pairs of studs per rib and single studs per rib.
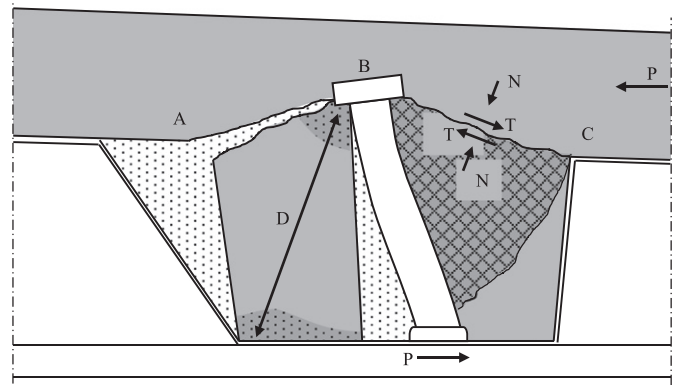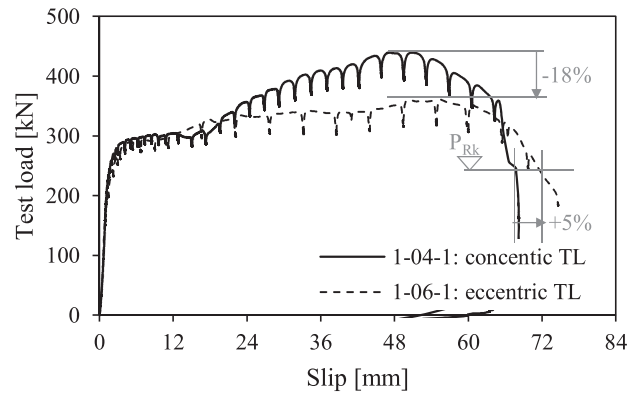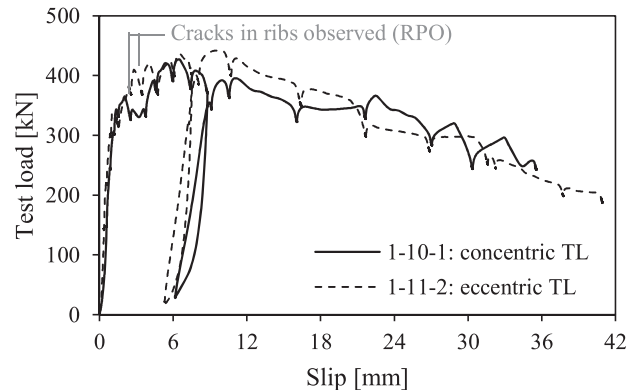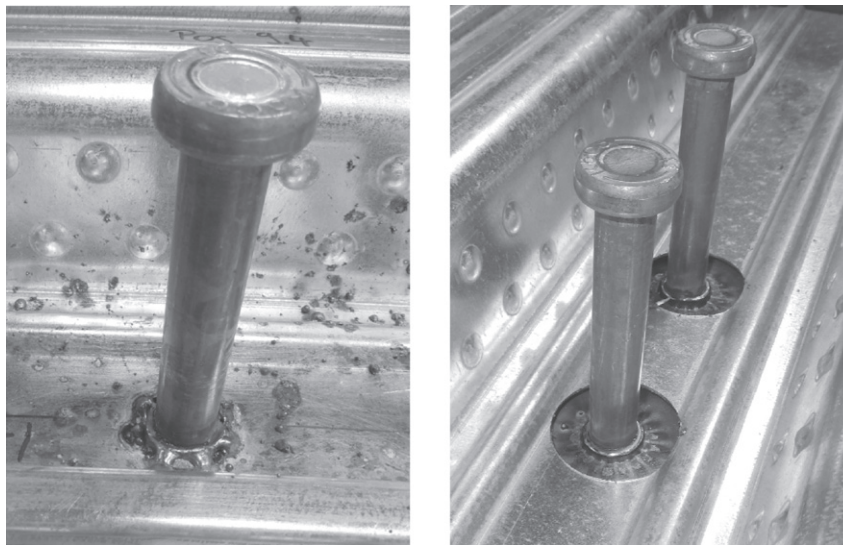

Fig. 31. Comparison of load–slip curves for tests with concentric and eccentric transverse loading (TL) for 80 mm deep decking.

(a) Through deck welded stud.

(b) Stud welded directly to the beam with pre-punched decking.

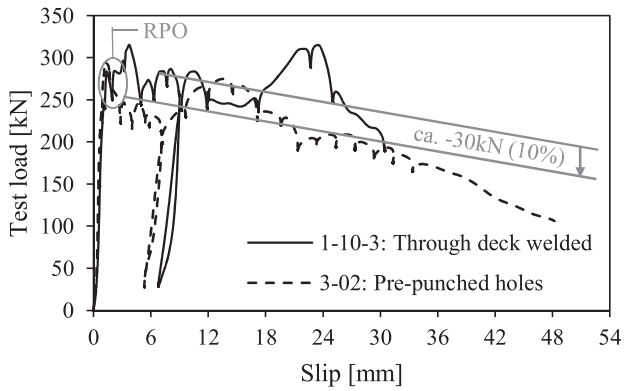Fig. 32. Through deck welded studs and studs welded directly to the beam with pre-punched decking.

Fig. 33. Comparison of the load–slip curves for pairs of studs welded through the decking and welded directly to the flange of the beam.
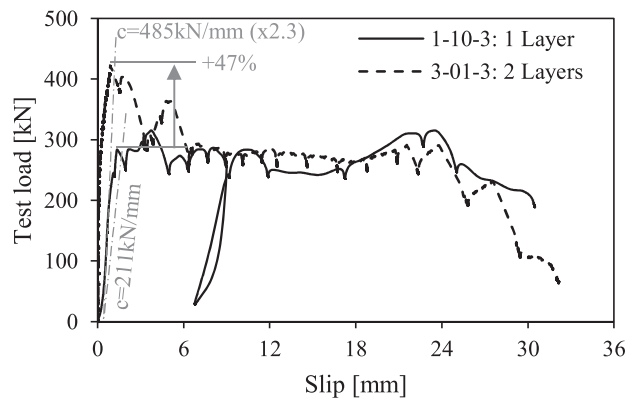

Fig. 35. Comparison of the load–slip curves for tests with one and two layers of reinforcement.

decking. The tension force is introduced into the slab at the re-entrant stiffeners on top of the sheeting (see Fig. 11). Because of the tension effect, a larger shear force was observed for through deck welded studs.

For through deck welded shear studs, the test load increased to a second load peak of about 300 kN at approximately 25 mm slip. This peak developed because the tension effect. The drop-off in the load
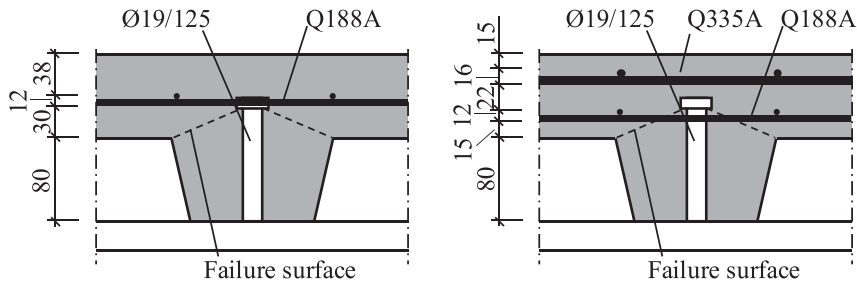
(a) 1 reinforcement layer

(b) 2 reinforcement layers

Fig. 34. Position of reinforcement for tests with one and two layers of reinforcement.

(a) 1 layer of reinforcement   (b) 2 layers of reinforcement

Fig. 36. Deformation of the push-out test specimen after testing.
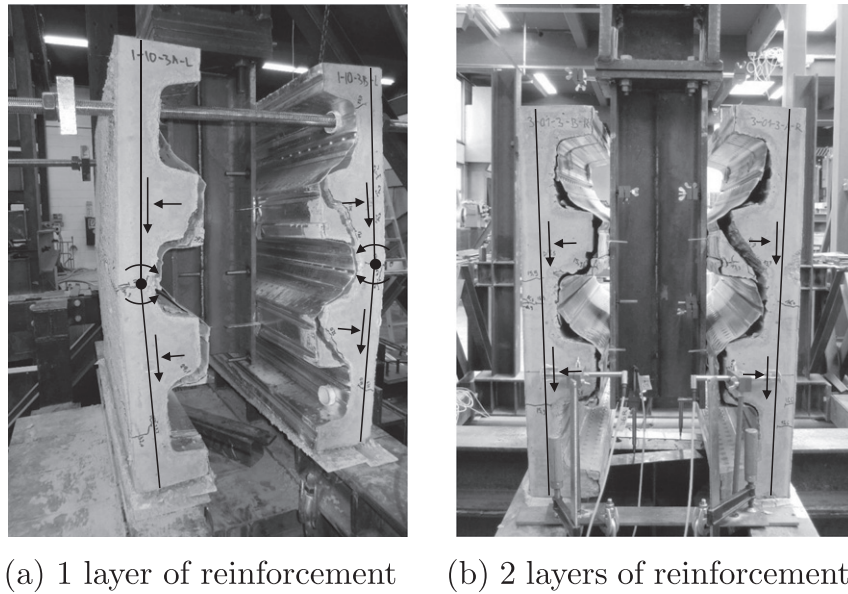
which followed was because of the debonding of the decking's re-entrant stiffeners from the concrete slab.

It can be assumed that shear studs with pre-punched decking show an in general 10% lower resistance than shear studs welded through the decking (see Fig. 33).

6.7. Influence of the number of reinforcement layers

For the specimen with two reinforcement layers, the bottom layer was placed 15 mm above the decking, and for the specimen with one reinforcement layer, it was placed 30 mm above the decking as shown in Fig. 34.

The number of reinforcement layers had a large influence on the stiffness and the failure load of the ribs, but not onto the post-failure behaviour (see Fig. 35).

Rib pry-out failure occurred in the test with a single reinforcement layer at a load of 287 kN and a slip of 1.4 mm. For the test with two reinforcement layers, rib pry-out failure occurred at a load of 422 kN and only 0.9 mm slip. This was an increase of the failure load of 47%. The stiffness of the shear connection was more than doubled. After the failure of the ribs, the load finally dropped to about 270 kN independently of the number of reinforcement layers.
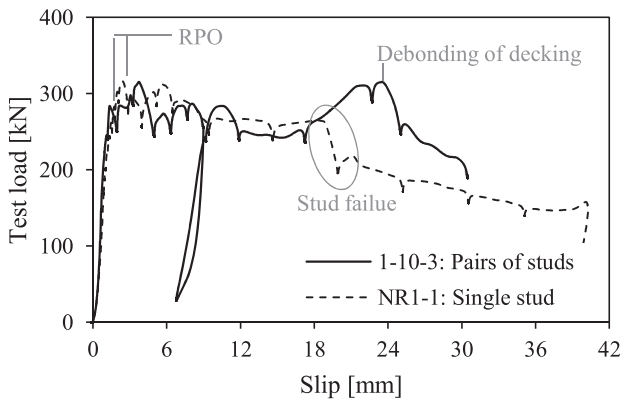
The lower position of the bottom reinforcement in specimen 3-01-3 improved the embedment conditions of the shear stud because the bottom reinforcement layer overlapped with the failure surface of the concrete cone. In addition, two layers of reinforcement led to a higher bending resistance and bending stiffness of the slab. Both details contribute to the increase of the failure load.

The influence of the number of reinforcement layers on the displacement behaviour of the slabs is shown in Fig. 36. For the specimen with only one layer of reinforcement, the slabs were subjected to higher bending displacements (see Fig. 36a). With two layers of reinforcement, the slabs did not bend but tilted over and rotated outwards at the support (see Fig. 36b). In both cases, the slabs were subjected to horizontal displacements because the force N, shown in Fig. 29, pushed the slab outwards when the failure surface slides along the concrete topping. As this global displacement was not restrained, there was no significant difference in post-failure behaviour (see Fig. 35).

6.8. Influence of the number of shear studs per rib

The influence of the number of studs per rib was only investigated for the 80 mm deep decking. As shown in the considerations on concentric transverse loading in Section 6.3, the number of shear studs per rib

Fig. 37. Comparison of load–slip curves of tests with 80 mm deep decking with single studs and pairs of studs per rib and no transverse load.
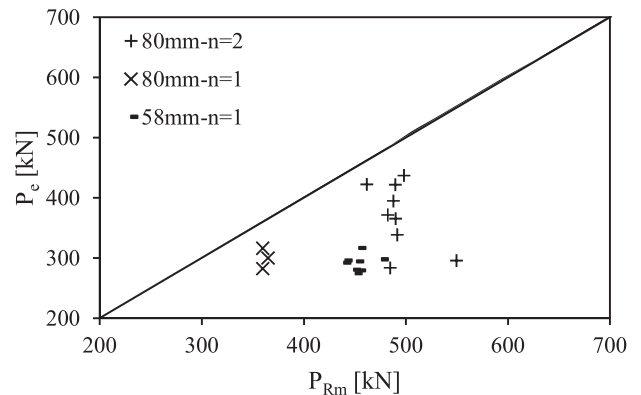

Fig. 38. Comparison of the measured resistances P_e with the analytical resistances P_{Rm} acc. to EN 1994-1-1 [2].
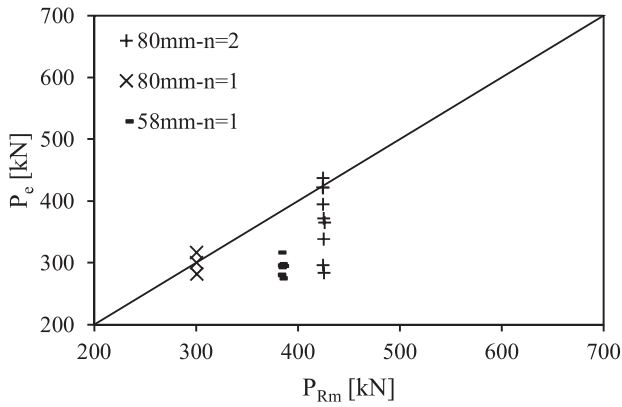
Fig. 39. Comparison of the measured resistance P_e with the analytical resistance P_{Rm} according to Lungershausen [1].
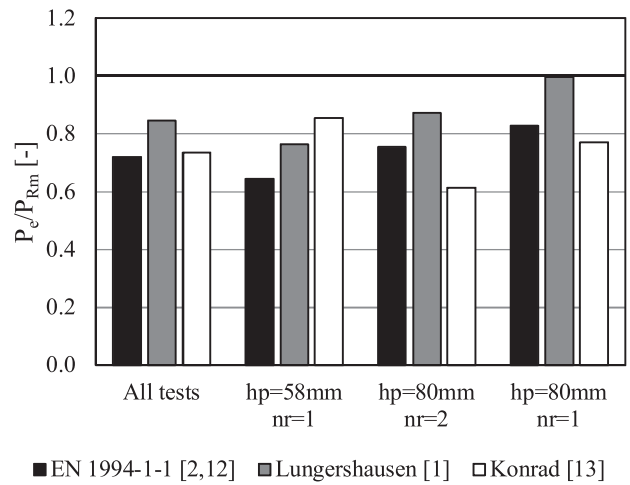

Fig. 41. Comparison of the ratios P_e/P_{Rm} for different analysis methods.

determined if the concentric loading was beneficial or not for this type of decking.

When considering tests without transverse loading, there was only negligible difference of the load–slip behaviour observed between single studs and pairs of studs per rib within the first 15 mm of slip (see Fig. 37). For single studs as well as for pairs of studs per rib, the observed failure mechanism was rib pry-out. The failure surfaces did not differ much from each other because the transverse spacing of the shear studs was small in comparison to the width of the failure cone (see Fig. 20). Therefore, the failure load of the ribs did not vary significantly.

Once a slip of about 15 to 20 mm was reached, significant differences were observed. For single studs per rib, steel failure occurred because of the higher loading per shear stud (see Fig. 28). In addition, there was no load peak due to the tension component of the decking because the shear studs punched through the decking.

7. Proposed testing procedure for push-out specimens with composite slabs

The presented push-out tests show that the influence of transverse loading is negligible in practice if the embedment depth of the head of the stud into the concrete topping is high.

However, for studs with a relatively small embedment depth, transverse loading significantly improved the load–slip behaviour and led to up to 30% higher shear resistances (see Table 5). No relevant influence of the loading procedure – constant or variable transverse loading – and the eccentricity was observed in the tests.
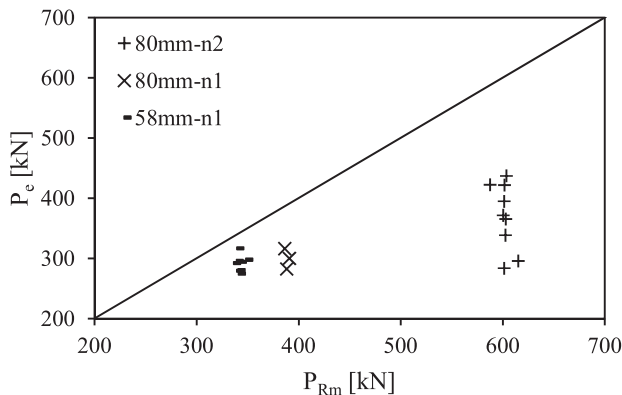

Fig. 40. Comparison of the measured resistance P_e with the analytical resistance P_{Rm} according to Konrad [11].

Based on these observations, it is recommended to conduct push-out tests as follows:

- With concentric transverse loading, when the embedment depth of the stud is too small to ensure double curvature of the headed studs.
- Without transverse loading, when the embedment depth of the stud is large enough to ensure double curvature of the headed studs.

As observed in the tests, the 2 diameters criterion of EN 1994-1-1 [2] is not sufficient to differentiate between a small and a large embedment depth. A more suitable criterion is given by Konrad [11]. Konrad numerically investigated the influence of the ratio of the stud height to the deck height h_{sc}/h_p onto the reduction factor k_t. It was found that the correlation curves for k_t changed at a ratio of $h_{sc}/h_p = 1.56$. This can be interpreted as a change of the failure mechanism and corresponds well with the observations in the presented push-out tests.

According to the results of the study presented in Section 2, a conservative value for the transverse load of 5% of the total test load is recommended. This is a degree of transverse loading per shear interface of 10%, which is slightly less than the value proposed by Hicks and Smith [4].

In general, the load-bearing capacity of the test specimen is not known. In this case, the procedure of a 'variable transverse load' shall be used. This means that during the test the transverse load must be permanently adjusted to maintain a value of 5% of the current vertical test load.

For a series of tests with nominal identical properties, variable transverse loading shall be used for the first test of the series. A value of 5% of the load-bearing capacity of the first test may be applied as a constant transverse load in further tests.

Table 7
Maximum reduction factors $k_{t,max}$ according to EN 1994-1-1 [2].

n_r	t [mm]	Through welded $d \le 20$ mm	Punched holes $19mm \le d \le 22mm$
1	≤1.00	0.85	0.75
	>1.00	1.00	0.75
2	≤1.00	0.70	0.60
	>1.00	0.80	0.60

8. Comparison with analytical resistances

8.1. Comparison with analytical resistances according to EN 1994-1-1 and Roik et al.

EN 1994-1-1 [2] assumes as basic failure modes either a failure of the stud (see Eq. (9) [15]) or a concrete compression failure directly in front of the weld collar (see Eq. (8) [15]). Both are failure modes of shear studs in solid slabs (see Fig. 1). To obtain the failure load of studs in slabs with trapezoidal decking, this resistance is multiplied with the factor k_t according to Eq. (10), which is assumed to be the mean value. Thereby, the reduction factor k_t is limited by the maximum reduction factor $k_{t,\mathrm{max}}$, shown in Table 7, as follows:

$$P_{m,c} = 0.374 \cdot \alpha \cdot d^2 \sqrt{f_c \cdot E_c} \tag{8}$$

$$P_{m,s} = 1.00 \cdot f_u \cdot \pi \cdot d^2/4 \tag{9}$$

$$k_t = \frac{0.7}{\sqrt{n_r}} \cdot \frac{b_m}{h_p} \cdot \left(\frac{h_{sc}}{h_p} - 1\right) \le k_{t,max} \tag{10}$$

$$P_{Rm} = k_t \cdot \min\begin{pmatrix} P_{m,c} \\ P_{m,s} \end{pmatrix} \tag{11}$$

where

f_c	concrete cylinder strength
f_u	stud tensile strength
E_c	Young's modulus of concrete
d	diameter of stud
b_m	rib width at mid-height of the deck profile
h_p	height of the deck rib
h_{sc}	as-welded height of stud
n_r	number of studs per deck rib

$e_{\mathrm{min}} = 2d$ minimum embedment depth in the slab
$\alpha = 1.0$ for $h_{sc}/d > 4$
$\alpha = 0.2 \cdot \left(\frac{h_{sc}}{d} + 1\right)$ for $3 \le h_{sc}/d \le 4$

However, a basic change of the failure mode, as has been observed in the tests, was not considered in EN 1994-1-1 [15,2].

The comparison between the average resistance according to EN 1994-1-1 [2] and Roik et al. [15] and the test results is shown in Fig. 38. It is shown that EN 1994-1-1 generally over-estimates test results.

For 58 mm deep decking, the width of the rib ($b_m = 81.5$ mm) is much smaller than for decks with comparable heights that are available in recent years [1,15]. For tests with 80 mm deep decking, the embedment depth of the head of the stud did not satisfy the minimum value of 2 diameters [2]. Because of this, the tested parameters match the limits of the database, which has been used in the calibration of EN 1994-1-1.

Table 8
Effective area of weld collar according to Konrad [11].

d	h_{Wulst}	d_{Wulst}	$A_{\mathrm{Wults,eff}}$
[mm]	[mm]	[mm]	[mm^2]
10	2.5	13.0	16.3
13	3.0	17.0	25.5
16	4.5	21.0	47.3
19	6.0	23.0	63.0
22	6.0	29.0	87.0
25	7.0	40.0	140.0

8.2. Comparison with analytical resistances according to Lungershausen

According to Lungershausen [1], the shear resistance of the shear connector is strongly dependent on the deformation behaviour of the shear stud itself. The resistance is derived from a load-bearing mechanism of the shear stud with two plastic hinges according to the plastic design theory (see Fig. 2). Accordingly, the mean value for the resistance of a stud is presented in Eq. (12), as follows:

$$P_{Rm} = 1.006 \cdot \frac{\beta}{\sqrt{n_r}} \frac{2 M_{pl}}{\tilde{a} \cdot d} \tag{12}$$

with:

$M_{pl} = \sigma_F \cdot d^3/6$ plastic bending resistance of stud
$\sigma_F = 500$ N/mm^2 nominal steel strength
$\tilde{a} = 0.8 \cdot \left(\frac{h_p}{b_o}\right)^2 + 0.6$ relative distance of hinges
b_o width of rib at its top
n_r number of studs per rib
$e_{\mathrm{min}} = 2d\sqrt{n_r}$ minimum embedment depth
β $\begin{cases} 1.00 & \text{for open deck shapes} \\ 1.10 & \text{for re-entrant deck shapes} \end{cases}$

The results of this comparison are shown in Fig. 39. The predicted resistances are also non-conservative in most cases.

For the test series NR1 with 80 mm deep decking and single studs per rib, the required embedment depth, e_{min}, was rarely satisfied but the shear resistance was well predicted with a test load of about 300 kN.

For series 1-04 to 1-06 with 58 mm deep decking, the load-bearing capacity is overestimated, even though the mechanism with two plastic hinges developed in the studs in the tests. The width of the decking is much smaller than for comparable decks used by Lungerhausen [1] to determine the distance between the plastic hinges in the stud. This means, that $\tilde{a}$ in Eq. (12) has not been calibrated for this narrow type of rib.

The tests with pairs of studs in 80 mm deep decking do not satisfy the required embedment depth, e_{min}, and a failure mechanism with only one plastic hinge developed (see Fig. 20). However, some studs in tests with higher transverse loading showed plastic deformations in the upper stud shank, but the cross-section cannot be assumed to be fully plastic. Thus, the load-bearing capacity is also overestimated.

8.3. Comparison with analytical resistances according to Konrad

The third study considers the reduction factors presented by Konrad [11]. The shear resistance of a stud in solid slabs is calculated according to Eqs. (13) and (14) and is reduced with reduction factors, which consider the geometry of the shear connection and the welding position.

Accordingly, the studs in 58 mm deep decking are in the unfavourable position and Eq. (15) is used. For 80 mm deep decking, the ratio h_{sc}/h_p is < 1.56 and Eq. (16) is used.

These equations have been derived for through deck-welded studs. They over-estimate the resistances for series 1-04 to 1-06 and 3-02. This effect may originate out of the use of a pre-punched decking. The Konrad formulae are as follows:

$$P_{m,c} = 39.5312 \cdot A_{\mathrm{Wulst,eff}} \cdot f_c^{2/3} + 3.72 \cdot d^2 \cdot f_c^{1/3} \cdot f_u^{1/2} \tag{13}$$

$$P_{m,s} = 38.2959 \cdot A_{\mathrm{Wulst,eff}} \cdot f_c^{2/3} + 0.57 \cdot f_u{}^2 \tag{14}$$

$$k_{\mathrm{unfav},3} = k_n \cdot \left[0.317 \frac{b_m}{h_p} + 0.06\right] \le 0.8 \tag{15}$$

$$k_{\mathrm{mid},1} = k_n \cdot \left[6.79 \cdot 10^{-4} \left(\frac{b_m}{h_p}\right)^2 + 0.170 \frac{b_m}{h_p} + 0.250 \frac{h_{sc}}{h_p}\right] \le 1.0 \tag{16}$$

where:

Table 9
Comparison of ratios P_e/P_{Rm} of different analysis methods for the prediction of the shear stud resistances.

		EN 1994-1-1	Lungershausen	Konrad
All tests	μ	0.721	0.846	0.734
	s	0.112	0.121	0.133
	V	0.155	0.143	0.181
58 mm-n = 1	μ	0.645	0.763	0.854
	s	0.031	0.035	0.040
	V	0.048	0.046	0.046
80 mm-n = 2	μ	0.754	0.871	0.615
	s	0.130	0.131	0.096
	V	0.172	0.150	0.155
80 mm-n = 1	μ	0.828	0.996	0.770
	s	0.048	0.058	0.046
	V	0.058	0.058	0.060

μ: average of the ratios P_e/P_{Rm}
s: standard deviation of the ratios P_e/P_{Rm}
V: coefficient of variation of the ratios P_e/P_{Rm}.

$A_{\text{Wulst,eff}}$ area of weld collar (see Table 8)
b_m rib width at mid-height
k^n $\begin{cases} 1.00 & \text{for single } studs \\ 0.80 & \text{for pairs of studs} \end{cases}$

The results of the comparison are shown in Fig. 40 and show that the equations of Konrad [11] over-estimate the resistance of the push-out tests in this paper.

The predictions according to Konrad [11] for specimens with 58 mm deep decking in series 1-04 to 1-06 are much better than EN 1994-1-1. This indicates the importance of considering the stud position because the equation for studs in an unfavourable position was used to calculate the shear resistance. The over-estimation could be because the decking was pre-punched prior to welding the studs.

The predictions for all tests with 80 mm deep decking show the largest deviation. Konrad [11] did not report a comparable failure mechanism for this deck height. Because of this, the reduction factor $k_{\text{mid},1}$ in Eq. (16) may not be accurate for rib pry-out failure, even though all requirements for the application of this factor are satisfied.

8.4. Comparison of the presented methods

A comparison of the results for all three analytical methods to predict the shear stud resistance is given in Table 9 and Fig. 41. If all 20 test results are considered, the method proposed by Lungershausen [1] is the most accurate. Significant differences between the methods of EN 1994-1-1 [2,15] and Konrad [11] were found when the stud position is considered. The method by Konrad gives the most accurate results for tests with 58 mm deep decking, where the studs are in unfavourable position because of the narrow deck rib, but predictions for tests with 80 mm deep decking are the most inaccurate of the three methods.

The results of the comparison show that the procedure of multiplying the resistance of a shear stud in a solid slab by an empirical derived reduction factor is insufficient to determine the resistance of a shear stud in composite slabs with deep profiles. The empirical reduction factors are only accurate within the range of parameters, which have been covered by the database of test results considered for the calibration of the reduction factor, k_t. The tests presented in this paper are not well covered by these databases—in terms of stud position, welding procedure and the observation of rib pry-out failure. The statistical evaluation of the influencing parameters [11] is not able to reflect significant changes in the load-bearing behaviour due to different failure modes.

Design procedures which are based on the stud shear resistance derived from mechanical models are better able to reflect failure modes, which were not covered by the statistical evaluation. This can be seen as the equation proposed by Lungershausen [1] shows the best results

in the presented comparisons. However, this equation also over-estimates the shear resistances. It is based on a simplified mechanical model and does not consider the concrete strength, the welding procedure or the stud position.

9. Conclusions

A series of 20 push-out tests with deep steel decking placed transversely to the steel beams was conducted. The investigated parameters were as follows

- shape and height of the composite deck profile
- number of reinforcement layers
- welding procedure
- degree of transverse loading
- influence of eccentric transverse loading.

The results of the presented push-out tests show that the load–slip behaviour and the failure mechanisms depend on the geometry of the deck profile and the embedment depth of the head of the studs in the concrete topping. The significance of the influence of transverse loading depended on the observed failure mechanisms. An eccentric application of the transverse load (to reflect negative moments in the slab) did not show a major influence on the load–slip behaviour, which would be relevant for design.

For shear studs with sufficient embedment depth, a load–slip curve with two load peaks develops, as described by Lungershausen [1]. In this case, the transverse load influences the load–slip behaviour similar to an increase in concrete strength, but the effect is relatively small.

With a small embedment depth, rib pry-out failure occurs at low slips. The observed influence of the transverse loading in the tests was inconsistent for single studs and pairs of studs per rib. For pairs of studs, the contact pressure at the failure surface was increased and led to a higher shear force. For single studs, the transverse load led to a failure of the compression struts in the failure cone or failure of the compressed face at lower shear forces.

Based on these results, it is recommended to apply a concentric transverse load of 5% of the total test load to the push-out specimen, if the height of the stud is less or equal to 1.56 times the deck height. Otherwise, transverse loading is not required.

The comparisons of the experimental and analytical resistances are non-conservative in most cases. The best correlation between the experimental and analytical resistances was obtained using the model according to Lungershausen [1]. The mechanical model is able to predict the shear resistance more accurately, even though the tested parameters were outside the considered range used by Lungershausen [1] in the calibration of the model.

The presented test results will assist in the development of improved design equations for the load-bearing capacity of stud shear connectors, where the additional failure mechanism rib pry-out, the influence of the transverse loading as well as the position of the shear studs have to be considered.

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