TECHNICAL ASSESSMENT

The behaviour and strength of steel-to-concrete connection using HVB shear connectors

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This technical assessment consists of 32 pages, 15 figures, and 5 tables. It has been prepared by Michel Crisinel, ing. dipl. EPFL/SIA.

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Prof. J.-C. Badoux

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1. GENERAL

1.1 Introduction

Shear connectors are elements which ensure steel-concrete interaction in composite construction. Their purpose is to transmit forces acting at the steel-concrete interface and to prevent uplift of the concrete slab from the underlying beam.

A new type of connector has recently been developed. Made of cold-formed steel and having an "L" shape, it is directly fastened to the underlying beam by two special fasteners using a powder-actuated tool.

The main advantage of this fastening method lies in the fact that no electrical power supply or welding is necessary.

This assessment contains the essential requirements for the use of HVB shear connectors in composite beams. A resume of tests, the values adopted for design, and rules concerning the positioning and placement of connectors are given. The assessment is based upon previous reports which are relevant for the connectors [1-5] and on the common unified rules for composite steel and concrete structures, Eurocode No. 4 [6].

1.2 Materials

1.2.1 HVB shear connector system

Connectors

The connector is made of cold-worked steel and has the general form of an "L". Three types of connector were tested.

Each type of connector has a different shape at the base and the top (FIGURE 1).

- HVB 80 (type 1)

- HVB 95/100 and HVB 105/110 (type 2)

- HVB 125 and HVB 140 (type 3)

The connectors are manufactured from St 4 LG steel (DIN 1624), which has a minimum specified yield strength of 200 N/mm^2. Tensile test specimens removed from the connectors had an average yield strength of 350 N/mm^2 and an average ultimate tensile strength of 355 N/mm^2. These high values result from strain hardening of the raw material during deep drawing the connectors.

Fasteners

ENP 3-21-L15 and ENPH3-21-L15 powder actuated fasteners are used to fasten the connectors to the underlying steel beam. These fasteners have shank diameters of 4.5 mm and are manufactured from spring steel. The factory certificate covering Hilti's

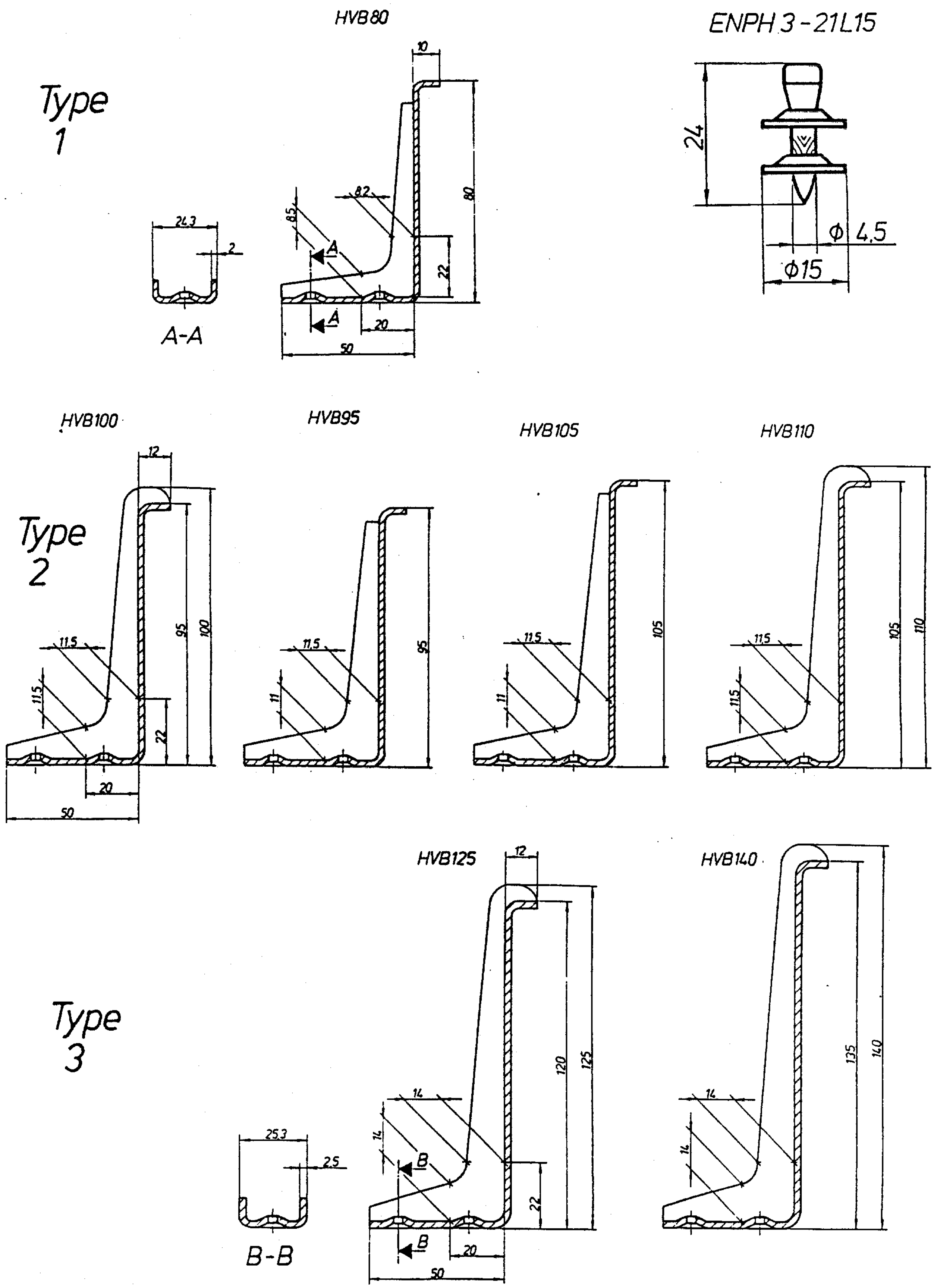

FIGURE 1 HVB shear connector - Types and dimensions

internal quality inspection shows the fasteners to have minimum Rockwell hardnesses of 55 (ENP) and 58 (ENPH), scale C.

Fastening tool

The connectors are fastened with a DX650 tool. The equipment must be used according to the recommendations published by the manufacturer.

Powder cartridges

The fasteners are driven using powder 6,8 x 18 M boosters, power level red or black, as recommended by the manufacturer.

Detailing and fixing

The detailing of the connector and the method of fixing it to the underlying beam is described in reference [7]. A typical detail is shown in FIGURE 2.

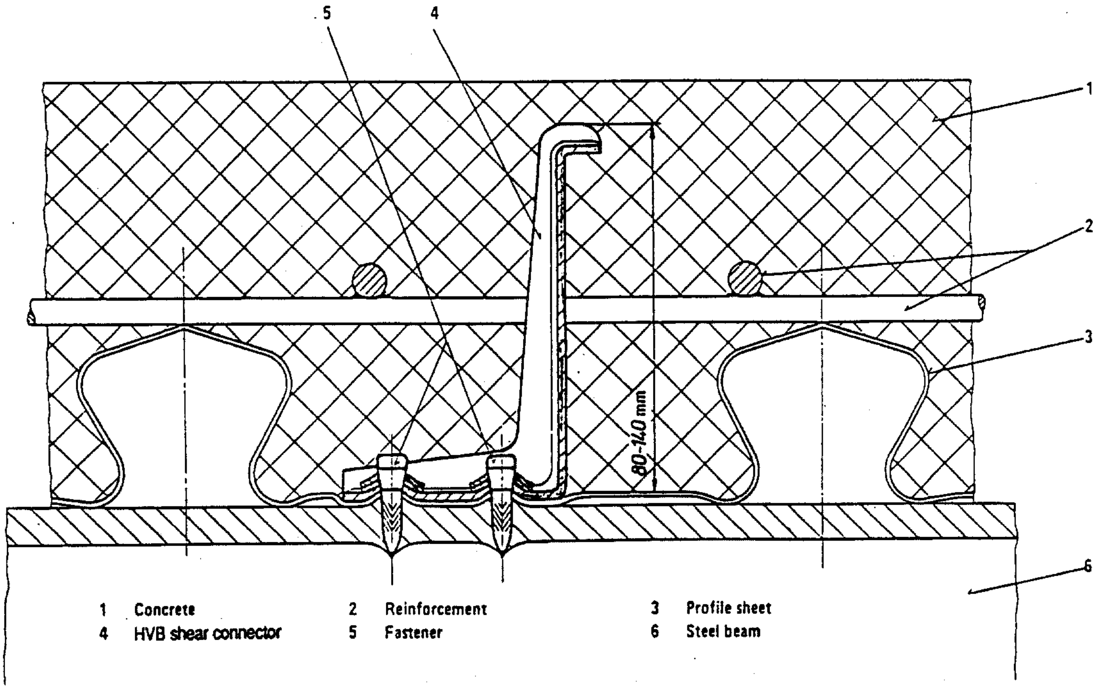

FIGURE 2 HVB shear connectors with profiled steel sheeting fixed to the underlying beam

1.2.2 Profiled steel sheeting

Push-out tests were performed using several different types of profiled steel sheeting. The purpose of these tests was to determine the influence of the sheet geometry on the behaviour and capacity of the connectors. The height of the profiled steel sheeting varied from 38 mm to 80 mm. The form of these sheets is shown in FIGURE 3. The sheets may be placed into three broad categories according to their height h_a and the slenderness of the profile (mean width of concrete ribs, b_o, and the pitch of the ribs in profiled sheet, b) :

- Compact profiles
  $b_o/b \cong 0.7 \div 0.8$, $b_o/h_a > 1.8$

  Examples : Cofrastra 40, Holorib 38, Holorib 51, Montarib 58

- Semi-compact profiles
  $b_o/b \cong 0.5 \div 0.6$, $b_o/h_a > 1.3$

  Examples : Hi-Bond 55, Cofrastra 70, Cobacier 80, US 2" LOK, US 3" LOK

- Slender profiles
  $b_o/b \leq 0.4$, $b_o/h_a > 0.8$

  Example : Haircol 80S

The thickness of the profiled steel sheet tested, varied between 0.75 mm and 1.0 mm. The yield stress varied between 220 and 320 N/mm^2.

1.2.3 Concrete

The concretes used for the tests conformed to the Swiss Code SIA 162, edition 1968 [8]. Three types of concrete with Rhine river aggregate, ϕ_{max} = 16 mm, have been used.

- normal concrete BN $\beta_{w28} = 20\ N/mm^2$
- high strength concrete BH $\beta_{w28} = 30\ N/mm^2$
- special concrete BS $\beta_{w28} = 60\ N/mm^2$

β_{w28} is the nominal 28 day concrete strength (value of compressive cube strength of concrete below which 1/6 of all possible test results would be expected to fall).

The concrete density was 2350 kg/m^3. Lightweight concrete was also produced for certain tests. Expanded clay aggregates (Leca hade) were used. The concrete complied with the specification of Directive 33 of the code SIA 162, edition 1974 [8]. The compressive strength varied between 30 and 50 N/mm^2 and the density was 1800 kg/m^3. For the analysis of results and the comparison with Eurocode 4 [6], the following relationship between the resistance in compression for a cube (β_w) and a cylinder (f_{ck}) has been taken as

$$f_{ck} \cong 0.85\ \beta_w$$

1.2.4 Steel beams

For both the composite beam and push-out tests, underlying beams of Fe 360 or Fe 510 steels, Euronorm 25 - 72 [9], were used. The type of section used for each test are given in the test reports [1-5].

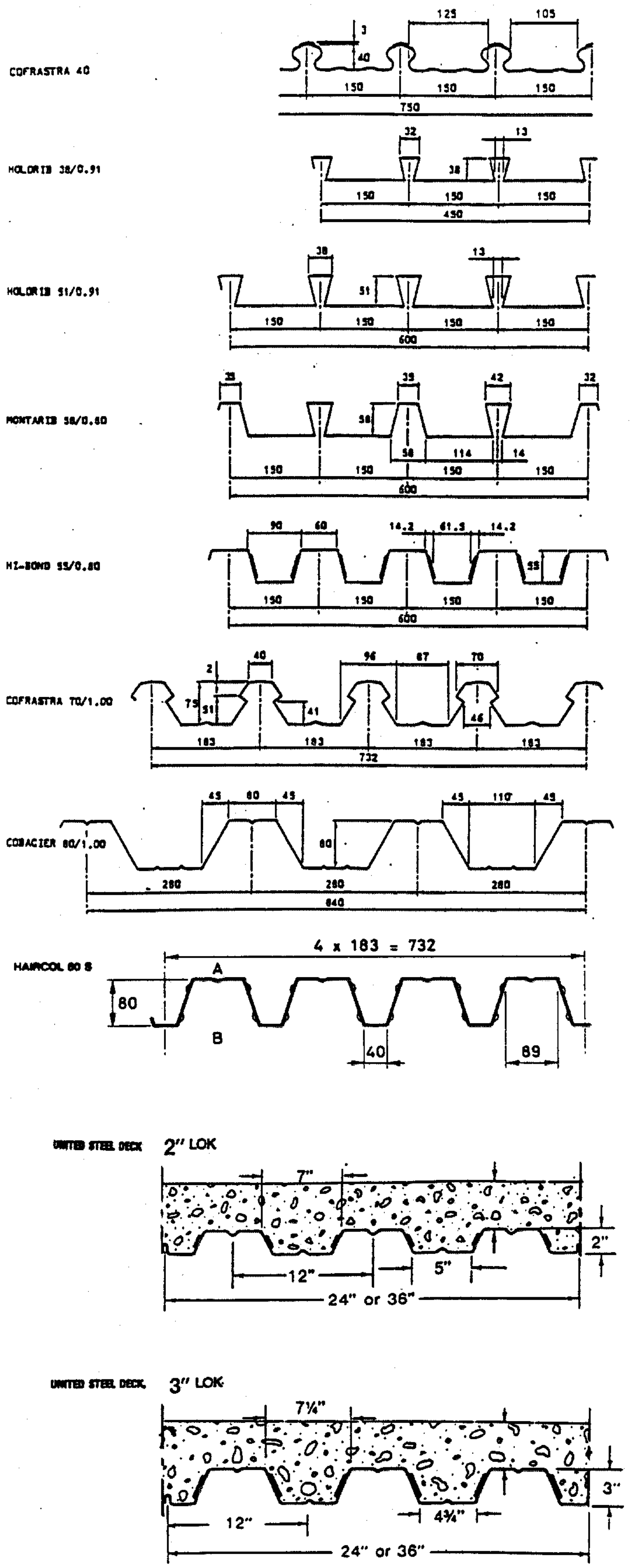

FIGURE 3 Profiled steel sheets

2. PUSH-OUT TESTS

Push-out tests were used to determine the influence of profiled steel sheets on the strength and the characteristic strengths for each connector type. Additional tests were conducted in order to determine the influence of the connector positioning (spacing, orientation with respect to the beam), the preliminary repeated loading and the concrete (normal-weight, light-weight or special concrete).

Push-out tests consisted of applying load to a centrally located steel beam, measuring the corresponding relative slip between the concrete slab and the steel beam, and observing the failure mode.

2.1 Test series

2.1.1 First series of tests

A series of 48 push-out tests was conducted in 1983 at Hilti Schaan (FL) under the control of ICOM (EPFL). These tests were based on the first connectors developed, HVB 80 and HVB 105 [1].

In addition to these tests a large number of similar tests were conducted in other countries, notably in France [12], England [13], Italy [14], and the United States [15]. In TABLE 1A a summary of variables studied is given together with the fixed parameters and the number of the tests conducted.

2.1.2 Second series of tests

The experience with the original types of connectors indicated that larger types should be developed. So, a second series of tests was carried out in 1987-1988 by Hilti at Schaan and by ICOM (EPFL) at Lausanne [2]. The TABLE 1B gives a summary of results obtained with normal and lightweight concrete. The details of the tests are given in the test reports [3, 5].

2.2 Test results

2.2.1 Push-out specimens with solid concrete slabs

Push-out tests were performed with solid reinforced concrete slabs to determine the characteristic connector strength without the presence of profiled steel sheets.

In FIGURE 4, all push-out test results are shown, grouped according to the connector height. P_{max} is the maximum experimental load per connector from the push-out test. The following parameters were studied :

TABLE 1A Push-out tests
First series performed during 1983 - 1984.

SERIES NO.	NUMBER OF SPEC.	CONNECTORS			CONCRETE	STEEL BEAM	PROFILED STEEL SHEET	PARAMETER STUDIED
		type	spacing a [mm]	Nr per specimen				
1.1	3	HVB 80	45	8	NW	IPE 180	Holorib 38	Type of profiled steel sheet
1.2	3	HVB 80	45	8	NW	IPE 180	Holorib 51	
1.3	3	HVB 80	45	8	NW	IPE 180	Montarib 58	
1.4	3	HVB 80	45	8	NW	IPE 180	Hi-Bond 55	
1.5	3	HVB 80	45	8	NW	IPE 180	-	
2.1	3	HVB 105	100	8	NW	HEB 240	Cofrastra 70	
2.2	3	HVB 105	100	8	NW	HEB 240	Cobacier 80	
2.3	3	HVB 105	100	8	NW	HEB 240	Montarib 58	
2.4	3	HVB 105	100	8	NW	HEB 240	Hi-Bond 55	
2.5	3	HVB 105	100	8	NW	HEB 240	-	
3.1	3	HVB 80	40	8	NW	HEB 240	Holorib 51	Spacing of the connectors
3.2	3	HVB 80	100	8	NW	HEB 240	Holorib 51	
4	3	HVB 80	45	8	NW	IPE 180	Holorib 51	Loading cycles
5	3	HVB 80	40	8	NW	IPE 180	-	Transverse loading
6.1	3	HVB 80	45	8	NW	IPE 180	Holorib 51	Concrete strength
6.2	3	HVB 80	45	8	NW	IPE 180	Holorib 51	

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NW : Normal-weight concrete

TABLE 1B Push-out tests
Second series performed during 1987 - 1988.

SERIES NO.	NUMBER OF SPECIMEN	CONNECTORS type	CONNECTORS spacing a [mm]	CONNECTORS Nr per specimen	CONCRETE	STEEL BEAM	PROFILED STEEL SHEET	PARAMETER STUDIED
1 2 3 4	3 3 3 3	HVB 140 HVB 140 HVB 110 HVB 110	184 184 184 184	8 8 8 8	NW NW NW NW	HEB 240 HEB 240 HEB 240 HEB 240	Haircol 80S - Haircol 80S -	Connector height Type of profiled steel sheet
5 6	3 3	HVB 100 HVB 100	184 180	8 8	NW NW	HEB 240 HEB 240	Hibond 55 Hibond 55	Connector positioning
7	3	HVB 100	184	4	NW	HEB 240	Hibond 55	Connector per rib
8 9 9-3 10	3 2 1 3	HVB 125 HVB 125 HVB 125 HVB 140	53 53 53 53	8 8 8 8	LW LW LW LW	HEB 240 HEB 240 HEB 240 HEB 240	- USD 3" USD 3" USD 3"	Connector positioning
Tests ICOM - Lausanne								
LS1-3 LD1-3 LD4-6	3 3 3	HVB 125 HVB 125 HVB 140	40/50 40/50 40/50	12 12 12	LW LW LW	HEB 240 HEB 240 HEB 240	- USD 3" USD 3"	Three connectors per rib
LD7 LD8 LD9	1 1 1	HVB 125 HVB 125 HVB 125	- 100	4 4 8	LW LW LW	HEB 240 HEB 240 HEB 240	USD 3" USD 3" USD 3"	Connector positioning

<u>Index</u>

LW : Light-weight concrete (ρ_o = 2350 kg/m^3)
NW : Normal-weight concrete (ρ_o = 1800 kg/m^3)

- Connector orientation with respect to the underlying beam (longitudinal or transverse)
- Concrete quality (normal or high strength)
- Concrete type (normal or light-weight)
- Number of connectors per rib and the distance between connectors (isolated or groups of connectors)

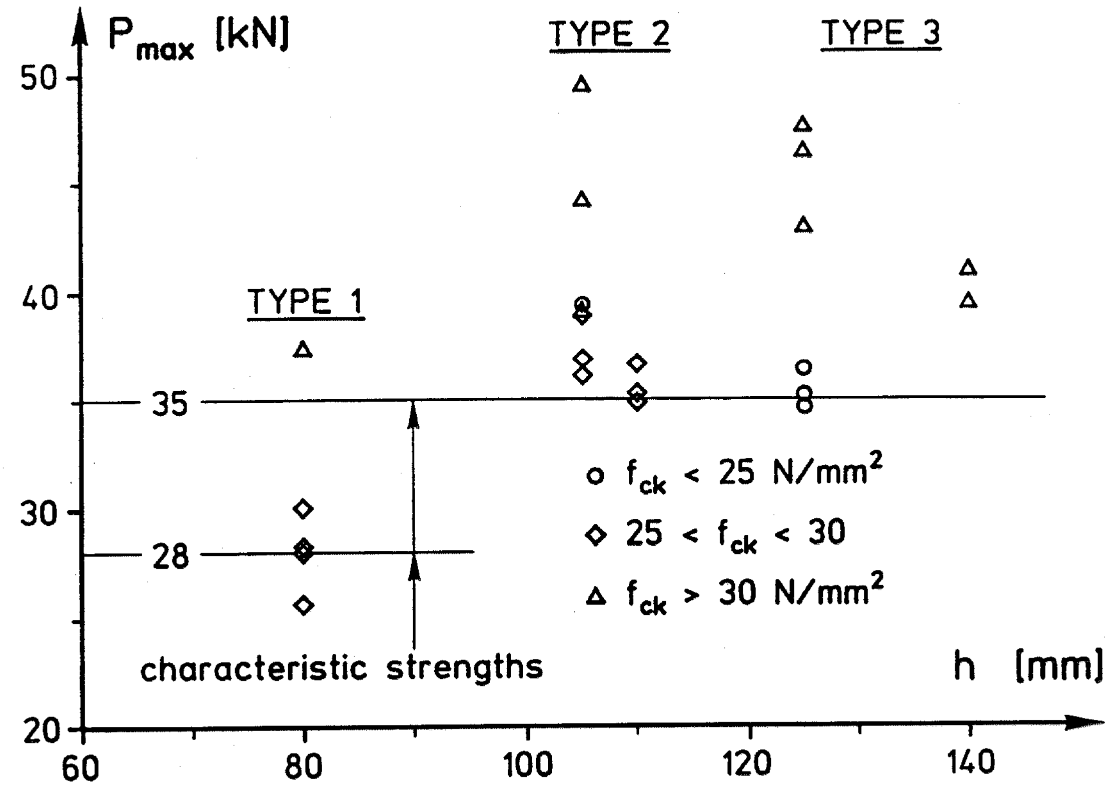


FIGURE 4 Influence of connector height and type on the characteristic connector strength for solid slabs

For the three connector types, the scatter was too large to distinguish the influence of each individual parameter. This large dispersion is due to the failure mode, either shearing or pulling out of the fastening pins (connector or concrete failure was never observed). The lowest strength for each connector type was taken as the characteristic connector strength. The characteristic connector strength so determined corresponds to the definition of the characteristic resistance as given in part 6.1.3 of Eurocode 4 [6]. A summary of these values is given in TABLE 2.

Ductile behaviour of connectors was observed for all tests conducted (FIGURE 5).

This ductile behaviour (indicated by the shape of the load / slip curve) is similar to that observed for tests performed using standard connectors (semi-automatically welded shear studs with enlarged heads).

TABLE 2 Characteristic connector strength

connector type	designation	Characteristic strength
1	HVB 80	P_{k1} = 28 kN
2	HVB 95/100 HVB 105/110	P_{k2} = 35 kN
3	HVB 125 HVB 140	P_{k3} = 35 kN

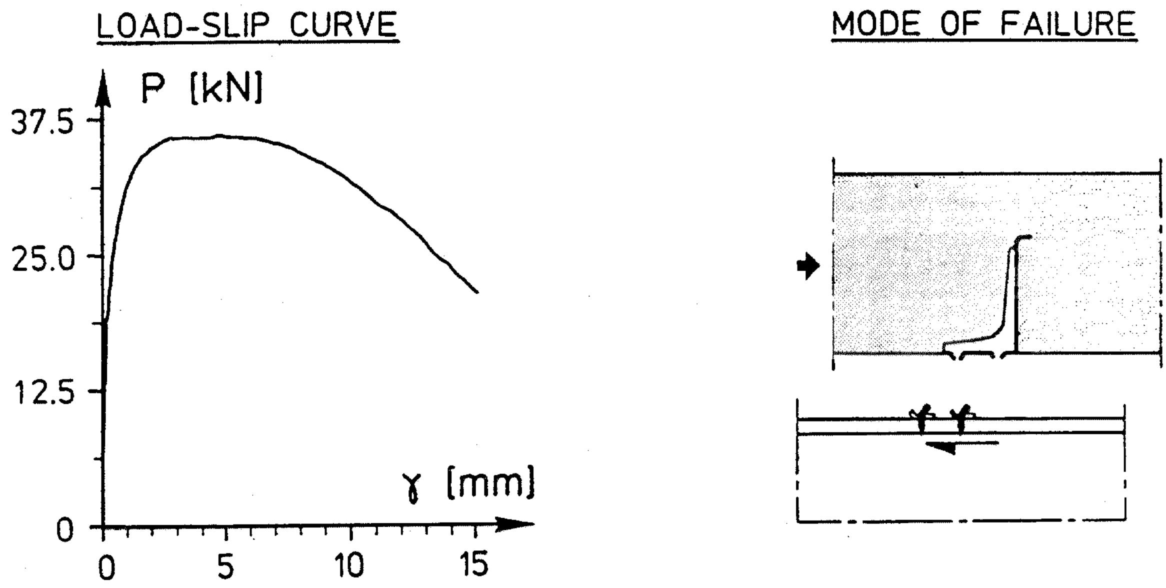


FIGURE 5 Load-slip diagram and failure mode of connectors with solid slabs

The measured slip is less than that observed for welded shear studs at the serviceability limit state. At the ultimate limit state large slips can occur between the concrete slab and the steel beam without greatly reducing the shear capacity of the connectors. The HVB shear connector can be considered ductile. The ultimate flexural capacity can be calculated according to plastic theory.

2.2.2 Push-out specimens with profiled steel sheet

The presence of the sheet may reduce the connector strength and may modify the behaviour at failure (FIGURE 6). In certain cases the deck may provoke brittle connector behaviour (FIGURE 6b). However in the majority of push-out tests with profiled steel sheet, semi-ductile behaviour was observed, that is to say, once the maximum load has been reached, connector strength decreases proportionally with slip (FIGURE 6a). This behaviour is also observed for welded shear studs with profiled steel sheet.

The similarity between both types of connector permits the use of the same reduction formula to predict the connector strength taking into account the deck geometry. This relationship is the following :

$$P_{k,rib} = r \cdot P_{k,sol} \qquad (1)$$

$$r = \frac{0.85}{\sqrt{N_r}} \cdot \frac{b_o}{h_a} \cdot \left(\frac{h}{h_a} - 1\right) \leq 1.0 \qquad (2)$$

$P_{k,rib}$: characteristic connector strength for slabs with profiled steel sheeting

$P_{k,sol}$: characteristic connector strength for solid slabs

N_r : number of connectors per rib

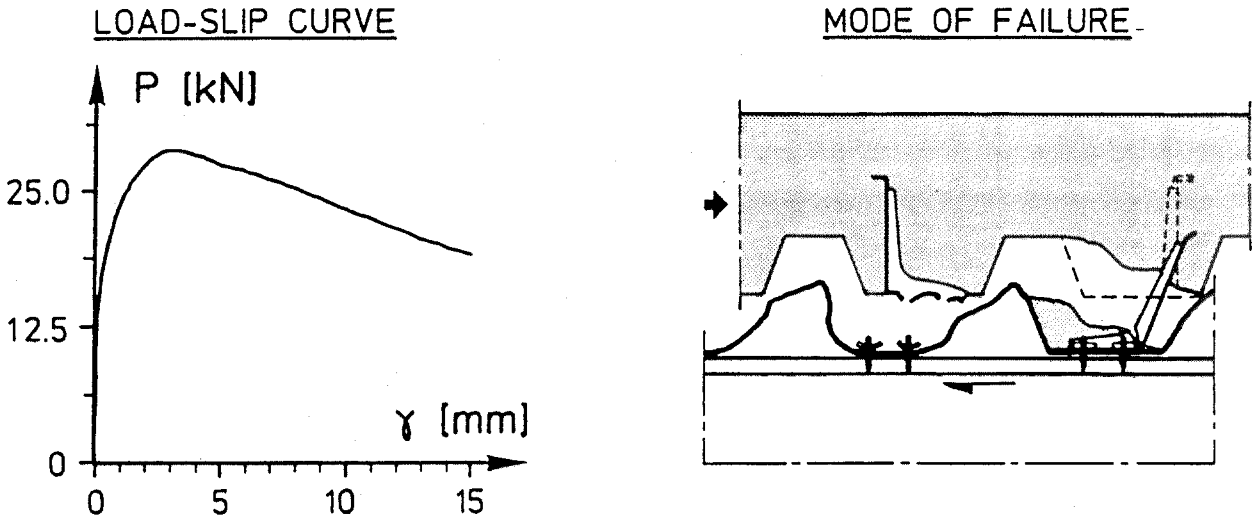
b_o : average rib width

h_a : rib height

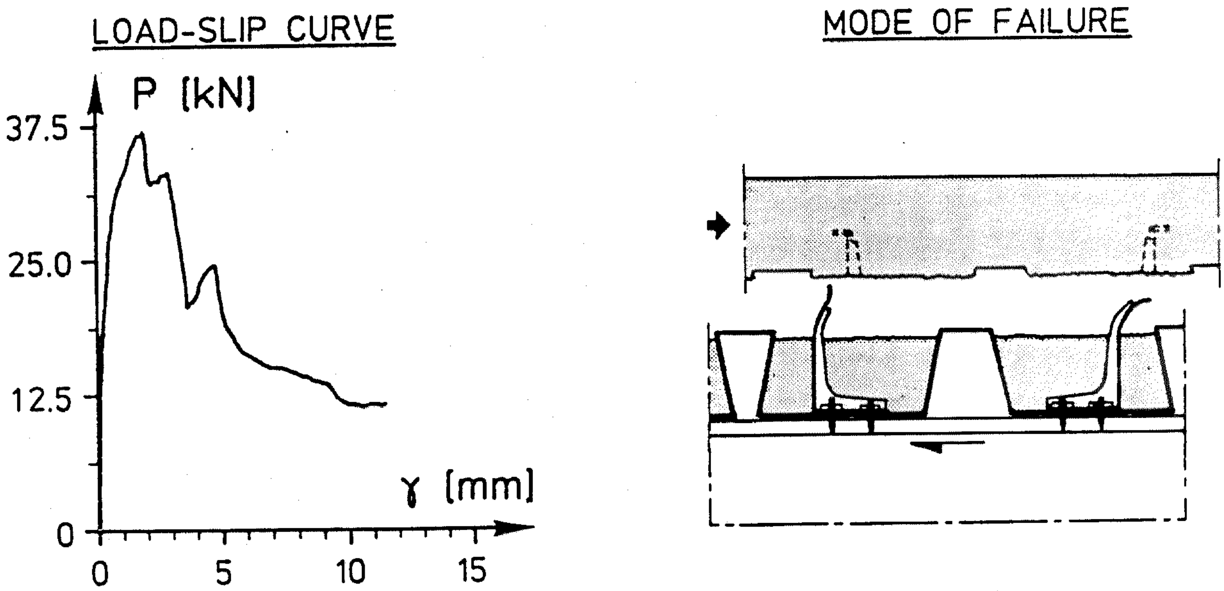
h : connector height

If $r \geq 1.0$, no reduction to connector strength , due to the presence of the profiled steel sheet should be made. The test results indicate that the reduction factor is not less than 0.5.

The variation between the results of the push-out tests and the values calculated using the reduction formula for test specimens with profiled steel sheet is shown in FIGURE 7. The experimental value P_{max} is divided by the theoretical value P_{th} (= $r \cdot P_k$) and presented as a function of h_a, the height of the profile. In order to establish detailing rules which ensures the ductility of the shear connection, a number of tests were carried out with the connectors in non-standard positions [2]. These results are represented by the solid symbols of FIGURE 7, which are below the line, $r = 1.0$. For these non-standard cases the reduction formula is not applicable. The detailing rules in chapter 5 ensures that these situations will not occur in practice.



a) Semi-ductile behaviour



b) Brittle behaviour

FIGURE 6 Load-slip behaviour and failure modes of connectors with profiled steel sheet

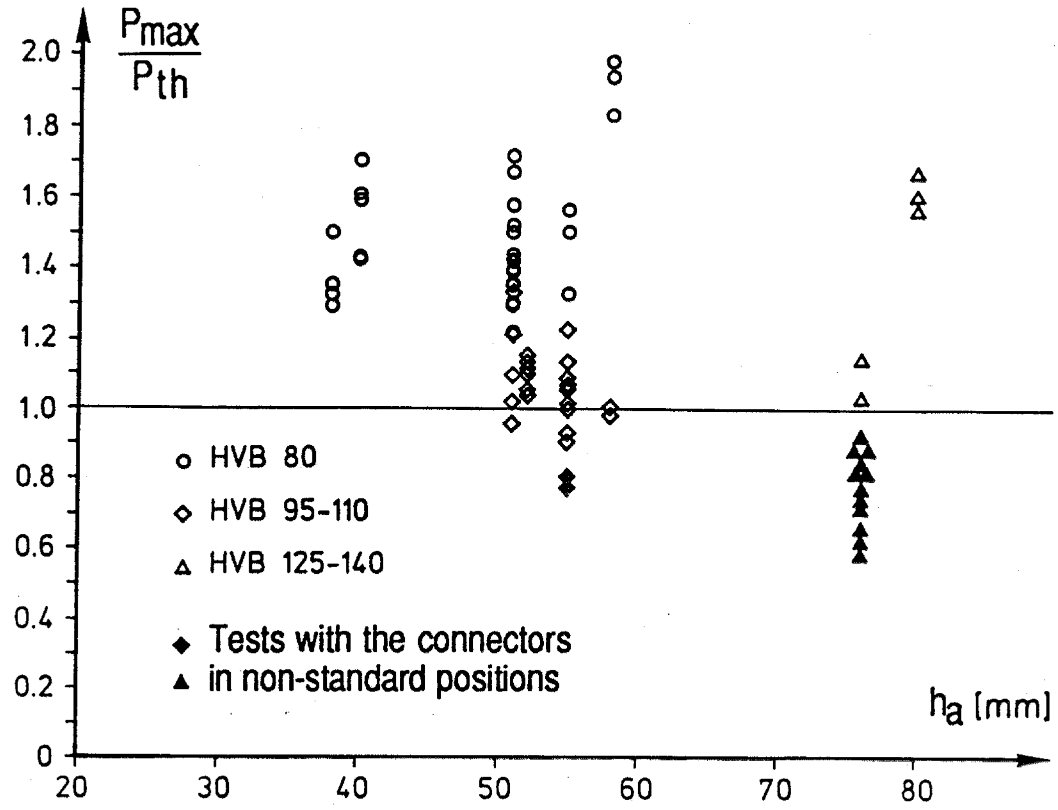


FIGURE 7 Variation in push-out test results with connector height compared to the theoretical value predicted by the reduction formula

The individual parameters which affect connector behaviour with profiled steel sheets are treated in detail in reference [1, 2]. The principal parameters are as follows :

Connector height above the rib, $h - h_a$

When $h - h_a > 35$ mm, the behaviour of the connectors during the tests was always ductile for the ratio $b_o/h_a \geq 1.8$. When $b_o/h_a < 1.8$ some brittle behaviour has been observed. These observations have enabled detailing rules to the established which ensure ductile behaviour of the connectors, an essentiel requirement for plastic design of composite beams.

Connector spacing

In the first tests performed with HVB 80 [1], the spacing had no influence on the connector capacity (two connectors per rib, spacing of 40 and 100 mm). The second series indicated a reduction of capacity as the spacing reduced (connectors HVB 125 and HVB 180, 75 mm deep profile, three connectors per rib, lightweight concrete). These observations enable the definition of minimum distances between connectors to be given to avoid a reduction in connector capacity and brittle failures.

Other influences

The following variables have also been studied. Their effect was not significant causing variations which were contained within the dispersion of the results. They are in particular :

- Cyclical loading (10000 cycles between 0 and 0.5 P_{max}). No significant influence.

- Concrete strength (compressive cube strengths from 22 to 60 N/mm^2). When β_{w28} was less than 35 N/mm^2 (f_{ck} < 30 n/mm^2) the shear connection behaviour was ductile. For certain specimens with high strength concrete the behaviour tended to be more brittle. The Eurocode 4 rule [6], which indicates that $f_{ck} \leq 30$ N/mm^2 for the connection to be considered ductile, is therefore equally applicable for HVB connectors.

- Lightweight concrete. Only structural lightweight concrete was used in the tests (specific density ≥ 1800 kg/m^3, characteristic cylinder compressive strength $f_{ck} \geq 25$ N/mm^2). No significant influence.

- Orientation of the connectors in relation to the beam (longitudinal or transverse).

  The tests with the connectors orientated in the two primary directions has not produced an exact relationship between the longitudinal and transverse capacities of the HVB connectors. In all cases, if the width of the ribs permit, place the connectors with the foot parallel to the beam.

In conclusion, the push out tests show that the HVB connectors behave in a similar manner to welded studs. When they are fixed in a solid slab their behaviour is ductile, the failure was due to shearing or pulling-out of the fasteners and not crushing of the concrete. They can therefore be considered ductile because their deformation capacity is sufficient.

When the connectors are fixed in a composite beam with profiled steel sheet, their behaviour is semi-ductile with the load-slip curve displaying a declining portion after reaching the maximum value. Certain detailing rules must therefore be respected (connector height above the ribs, distance between the connectors, distance from the profile ribs and the profile height to width ratio) in order to assume ductile behaviour in design. Because the HVB connector behaviour is similar to that of welded connectors, the same reduction formula may be applied to determine the effect of the profiled steel sheet.

3. COMPOSITE BEAM TESTS

3.1 Specimen description

Three composite beams with HVB shear connectors were tested. The beams, which were the same except for the number of connectors used, were made using IPE 220 hot rolled sections in Fe 510 steel. The span length was 6 m and the slabs were 2 m wide and 120 mm thick. The profiled steel sheet was Hibond 55 (0,75 mm thick) with the ribs (150 mm spacing) perpendicular to the beam.

The beams were fully supported during concreting.

The following numbers of shear connectors, placed transversally with respect to the steel beam, were used :

- 2 HVB 100 connectors per rib. This corresponds to a theoretical degree of partial connection of 56%.
- 1 HVB 100 connector per rib. A connection of 40%.
- 1 HVB 100 connector every other rib. A connection of 20%.

In FIGURE 8 the composite beam dimensions are shown.

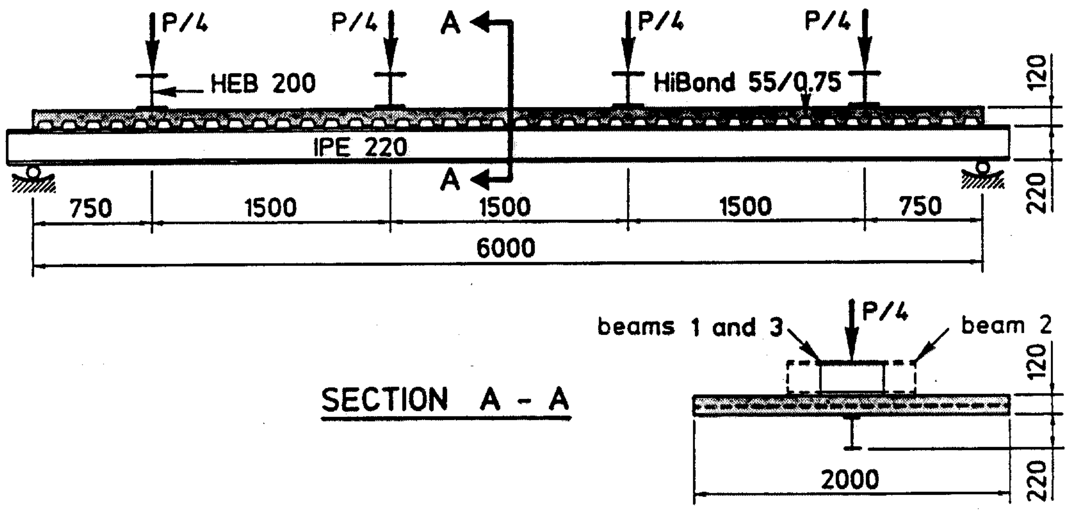


FIGURE 8 Composite beams - dimensions and load locations

3.2 Test procedure

Each simply supported composite beam was subjected to four line loads placed at 1/8, 3/8, 5/8 and 7/8 of the 6 m span (FIGURE 8). The loads were displacement controlled. The four corners of the slab at the support were supported during testing to avoid end rotations of the beam. The load steps were approximately 10 kN, the readings were taken after the loading stabilized. The following measurements were made :

- midspan displacement of the beam
- vertical slab displacement (10 locations)
- slip between the formed steel deck and the slab (7 locations)
- strains in the underlying beam (2 locations)
- strain in the concrete slab (1 location)

3.3 Test results

A detailed summary of all test results is given in the Hilti report - IB 04/88 March, 1988 [4]. A resume of the test results is shown using the following:

- Load - Displacement curves (FIGURE 9).
- Load - Slip curves (FIGURE 10).
- Values given in TABLE 3.

TABLE 3 Composite beam test results

Test number	Number of HVB100 connectors	Degree of connection %	P_{max} kN	M_g kNm	M_p kNm	$M_{u,test}$ kNm
1	20	20	267	22	200	222
2	80	56	346	22	260	282
3	40	40	306	22	228	250

P_{max} : maximum applied load
M_g : dead load moment
M_p : maximum moment due to P_{max}
$M_{u,test}$: ultimate experimental moment ($M_{u,test} = M_g + M_p$)

Concrete cube strengths, β_w = 36.7 N/mm^2 were obtained. The measured yield stress of the steel, σ_f, was 460 N/mm^2.

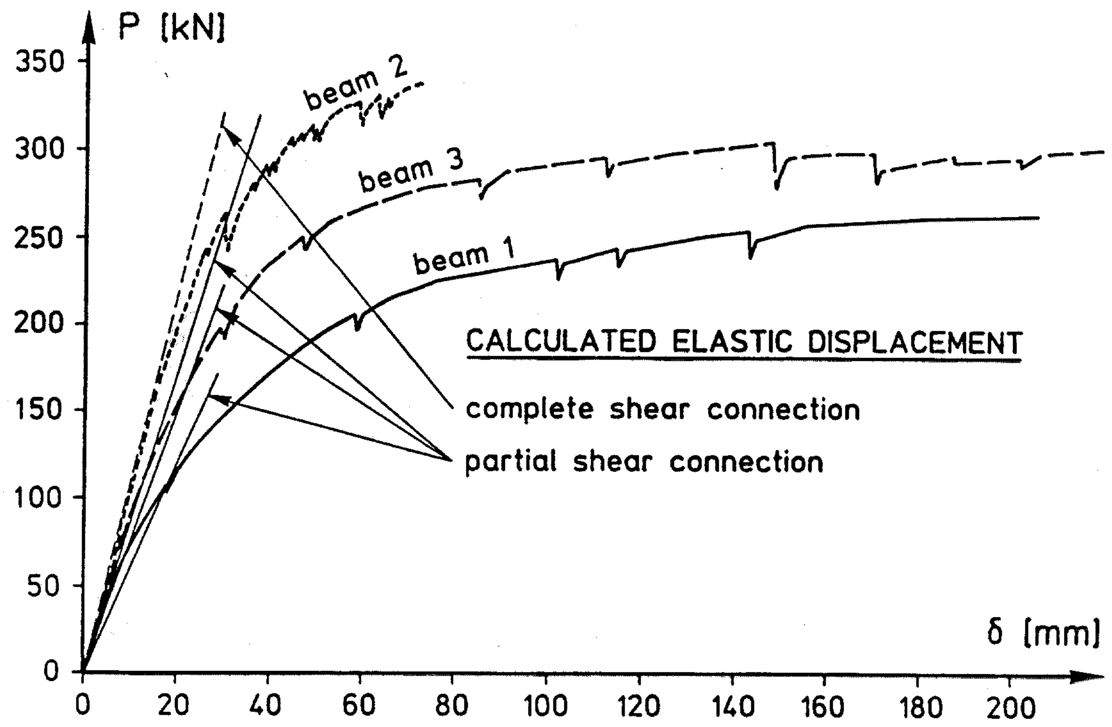
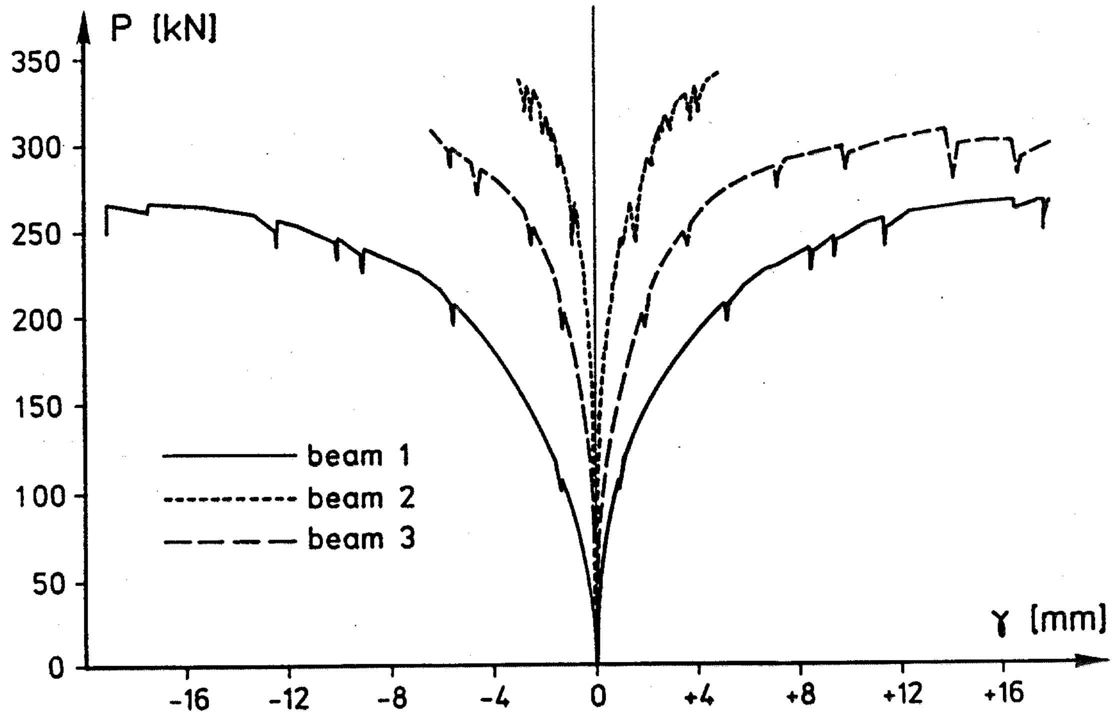


FIGURE 9 Load/mid-span displacement relationship (measured and calculated)



The composite beam flexural capacity is represented graphically as a function of the degree of connection in FIGURE 11, in which :

$M_{p,a}$: ultimate flexural capacity of the underlying beam alone
$M_{p,f}$: ultimate flexural capacity of the composite beam with full connection
M_p : ultimate flexural capacity of the composite beam with partial connection
N : number of connectors in the composite beam tested
N_f : number of connectors necessary to ensure full connection.

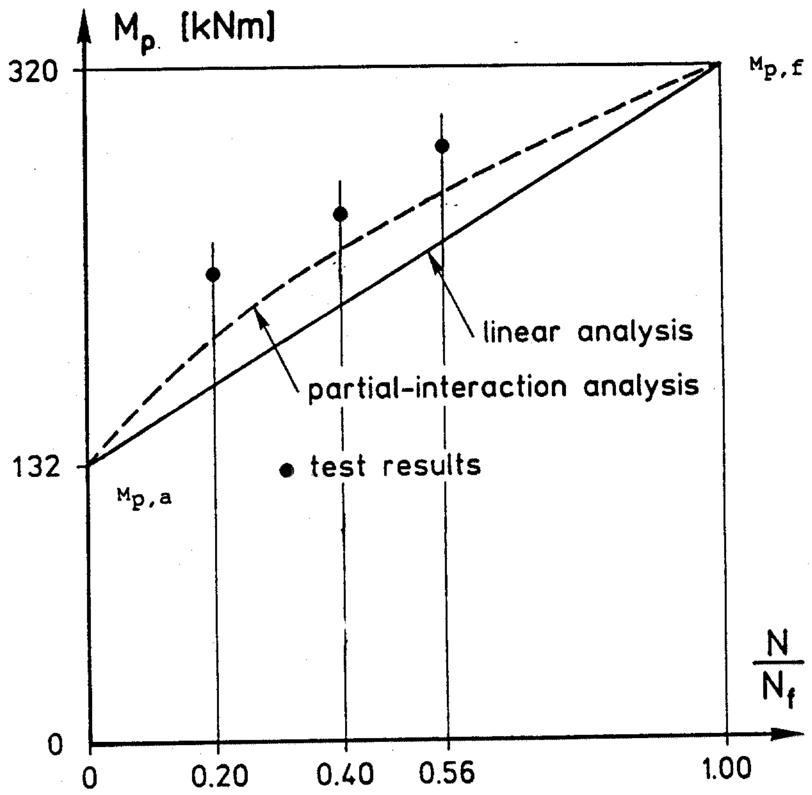


FIGURE 11 Influence of the degree of connection on the composite beam flexural capacity

3.4 Conclusions

The composite beam tests with partial connection indicate that behaviour remains ductile even with low degrees of connection. Composite beam strengths may be conservatively calculated using a simple analytical model without taking slip into account.

4. DESIGN OF COMPOSITE BEAMS WITH HVB SHEAR CONNECTORS

The design of composite beams in buildings using steel beams and a concrete slab with or without profiled steel sheets and HVB shear connectors may be carried out in the same manner as that employed for welded shear studs.

In Europe, the calculation of composite beams will be specified in the future by Eurocode 4 [6]. The design for the composite action with HVB connectors can be based on these rules provided the following conditions are followed.

4.1 Materials and installation

The fastening using HVB connectors must correspond to the specification of Hilti AG [7]. This describes the shape and material of the connector, and the method and checking of the fastening.

4.2 Horizontal shear force

The connectors must be distributed throughout the length of the composite beam to transmit the horizontal force between the concrete slab and the steel beam.

For plastic design, the ultimate flexural capacity may be determined plastically. The horizontal force is determined by considering the equilibrium of the elements of the beam at the critical sections (sections between zero moment and maximum plastic moment).

In the zone of positive moment, the ultimate horizontal force is the smaller of the following :

$$F_u^+ = f_y \cdot A_a/\gamma_a \tag{3a}$$

$$F_u^+ = 0.85\ f_{ck} \cdot A_c/\gamma_c \tag{3b}$$

f_y/γ_a : design yield stress for steel
A_a : area of steel beam
$0.85\ f_{ck}/\gamma_c$: design compressive strength of concrete
A_c : effective area of concrete slab.

In the zone of negative moment, where the continuity of the composite beam is ensured by reinforcing steel in the slab, the ultimate horizontal force is given by :

$$F_u^- = A_s \cdot f_{ys}/\gamma_s \tag{4}$$

A_s : area of reinforcing steel in the effective width
f_{ys}/γ_s : design yield stress of reinforcing steel.

In the case of elastic design, the calculation of the connection is based on the elastic horizontal force :

$$v = \frac{V \cdot S_c}{n \cdot I_b} \tag{5}$$

v	: elastic horizontal force per unit length
V	: shear force
S_c	: first moment of area of the slab with respect to the neutral axis
n	: modular ratio steel/concrete $(n = E_a/E_c)$
I_b	: second moment of area of composite beam

The distribution of the connectors is made on the basis of the shear force diagram.

4.3 Connector strength

The strength of the connectors, determined experimentally using push-out tests (see chapter 2) is given in TABLE 4 below. The characteristic strength and the design values are independent of the type of concrete (normal or lightweight). The HVB connectors are flexible provided the characteristic compressive cylinder strength of concrete does not exceed 30 N/mm^2. Their deformation capacity is sufficient to justify the assumption of ductile behaviour of the connection.

TABLE 4 HVB Connector strength

HVB type no	Height h [mm]	Characteristic Strength P_k [kN]	Design Strength P_d [kN]
1	80	28	23
2	95/100 105/110	35	28
3	125 140	35	28

4.4 HVB connectors with profiled steel sheet

When the connectors are fixed into a steel beam through the profiled steel sheet of a composite slab, the strength of the connector may be affected.

If the ribs of the profile are perpendicular to the beam, the values given in paragraph 4.3 must be multiplied by the following reduction factor :

$$r = \frac{0.85}{\sqrt{N_r}} \cdot \frac{b_o}{h_a} \cdot \left(\frac{h}{h_a} - 1\right) \leq 1.0 \qquad (6)$$

N_r : number of connectors distributed in a rib, up to a maximum of three
b_o : average width of concrete rib
h_a : profiled steel sheet height
h : connector height.

When the ribs of the profile are parallel to the beam, the strength values given in table 4.3 are applicable subject to the ratio $b_o/h_a > 1.8$. If $b_o/h_a \leq 1.8$ the values must be multiplied by the following reduction factor

$$r = 0.6 \frac{b_o}{h_a} \left(\frac{h}{h_a} - 1\right) \leq 1.0 \qquad (7)$$

4.5 Required number of connectors

The number of connectors between two critical sections (for example a section of maximum moment, positive or negative, and a section of zero moment) must be at least equal to the ultimate horizontal shear force divided by the design strength of a connector.

$$N = \frac{F_u}{P_d} \qquad (8)$$

F_u : horizontal force determined according to (3) or (4)
P_d : design value according to Table 4, reduced where necessary by the factor r given by formulae (6) or (7)

To avoid uplift of the concrete slab, it must be attached to the beam by the connectors at a longitudinal spacing not exceeding 400 mm. When the slab is formed with a profiled steel sheet, the attachements (at a spacing of 400 mm maximum) may be :

- HVB connectors
- HVB connectors alternating with ENP pins
- other means of attachement

4.6 Partial connection

If the number of shear connectors is less than the number determined by paragraph 4.5, the connection of the composite beam is partial. According to Eurocode 4 [6], the number of connectors must not be less than 50 % of the number required for full connection to satisfy the strength criteria. The HVB connectors are sufficiently ductile so that this rule is also applicable. The calculation of the ultimate moment of resistance of the composite beam with HVB connectors in partial connection can be carried out according to Eurocode 4 [6].

The tests carried out on three composite beams with partial connection (see chapter 3) have shown that safety was ensured in the three cases. A degree of connection less than 50 % can therefore be allowed provided it is based on a scientific design method (for example a numerical method of analysis [10]) or shown by tests that the safety criteria are met.

If the partial connection is required to control deflection of the steel beam only, the degree of connection may be reduced to as low as 25 %. In this case, it is necessary to show by calculation or by test, that the steel beam alone is capable of resisting all the loads and that there is not a premature rupture of the connectors by excessive slip.

4.7 Service performance

The Eurocode 4 serviceability requirements [6] for the verification of service performance of composite beams (concrete cracking, deflection and vibration) are applicable for beams using HVB connections. The calculation of deflection of beams with partial connection can be carried out using the second moment of area given by the following formula [11] :

$$I = I_a + \sqrt{\frac{N}{N_f}}\,(I_f - I_a) \tag{9}$$

I	: Second moment of area of composite section with partial connection
I_f	: Second moment of area of composite section with full connection
I_a	: Second moment of area of steel beam alone
N/N_f	: Degree of partial connection.

Alternately, the deflection of partially connected beams can be obtained from the following formulae [16] :

$$\delta = \delta_f + 0.5\,(1 - \frac{N}{N_f})\,(\delta_a - \delta_f) \text{ for propped beams} \tag{10}$$

$$\delta = \delta_f + 0.3\,(1 - \frac{N}{N_f})\,(\delta_a - \delta_f) \text{ for unpropped beams} \tag{11}$$

δ_f	: Deflection of the composite beam with full connection
δ_a	: Deflection of the steel beam alone.

5. POSITIONING AND INSPECTION OF HVB CONNECTORS

5.1 Connector positioning

5.1.1 Solid slabs

Push-out tests with HVB shear connectors embedded in solid concrete slabs have shown that their strength and behaviour is not affected provided the following conditions are met :

a) Connector spacing (between vertical leg axes)

- Minimum transverse connector spacing is 50 mm
- Minimum longitudinal connector spacing is 100 mm
- Maximum longitudinal connector spacing is the smaller of the following :
  - $4h_c$ (h_c = slab thickness)
  - 600 mm

b) Connector orientation

The connectors may be placed either parallel or perpendicular to the underlying beam

c) Concrete type and quality

The specified characteristic cylinder strength f_{ck} is not greater than 30.0 N/mm^2 (normal or light-weight).

d) Beam flange thickness

The minimum flange thickness of the underlying beam is 8 mm.

5.1.2 Slabs with profiled steel sheet

The connector strength and behaviour may be affected by the geometry of the profiled steel sheet and the connector location relative to the ribs. When HVB connectors provide the connection between a composite slab and a steel beam the following rules given in 5.1.1 must be followed. Also the following rules must be applied to take into account the geometry of the profiled steel sheet.

a) Distance between connectors

- Profiled steel sheet ribs perpendicular to the beam.

  The transverse distance between the axes of the vertical legs is not to be less than :

  - compact profiles : 50 mm
  - other profiles : 100 mm

  The longitudinal distance between the axes of the vertical legs is not to be less than the pitch of the ribs.

- Profiled steel sheet ribs parallel to the beam

  Longitudinal distance : see 5.1.1 a)

  In the transverse direction :

• compact profiles with	$b_o \geq 100$ mm	: maximum 2 connectors
• other profiles	- $b_o \geq 60$ mm	: maximum 1 connector
	- $b_o \geq 100$ mm	: maximum 2 connectors

b) Distance between the connectors and the web of the profiled steel sheet rib

- Profiled steel sheet perpendicular to the beam.

  If one connector per rib :
  The distance between the vertical leg of the connector and the mid-height of adjacent web must not be less than 40 mm.

  If two connectors per rib :
  They should be placed centrally in trough or alternately off centre.

- Profiled steel sheet parallel to the beam.
  The distance between the vertical leg of the connector and the closest part of adjacent web must not be less than 20 mm.

c) The height of the connectors above the ribs

If $b_o/h_a \geq 1.8$, the length $h - h_a \geq 35$ mm.
If $b_o/h_a < 1.8$, the length $h - h_a$ must be at least the larger of the two following values

$$h - h_a \geq 35 \text{ mm}$$

$$h - h_a \geq \frac{2}{3} h_a$$

d) Presence of a stiffener in the bottom flange of the profiled steel sheet

When the connectors are to be fixed through the bottom flange of a sheet and the bottom flange has a longitudinal stiffener then the following rules must be followed :

- small stiffener ($h_r \leq 3$ mm) : the stiffener may be flattened with a hammer (or similar) prior to fixing the connector.

- large stiffener ($h_r > 3$ mm) : if it is not possible to flatten the stiffener, the connectors must be placed at the side of the stiffener (minimum two connectors placed alternately off centre).

In all cases, the minimum distances described in a) and b) must be complied with.

The FIGURES 12 to 15 following give a summary of the rules described above.

5.2 Inspection of connectors

The following details must be checked on the construction site when using HVB connectors.

a) Connector positioning

It is necessary to check that the rules of paragraph 5.1 are complied with.

b) Fastener installation (adequate penetration into the steel beam)

The fastener penetration can be checked measuring the exposed fastener height. This distance should be between 6.5 and 9 mm. Visually, the connector should be firmly seated against the profiled steel sheet but not deformed.

The connectors **should not** be checked in the same manner as for welded shear studs by bending at an angle. This procedure does not represent the load condition to which the connector is subjected in service.

6. CONCLUSIONS

The tests on the push-out specimens and on the simply supported composite beams have shown that

- the composite action between steel and concrete can be satisfactorily acheived using HVB connectors
- the slab can be reinforced concrete or a composite slab with a profiled steel sheet
- the concrete can be normal or lightweight

If the conditions and rules of construction of Chapter 5 are followed, the connection by HVB connectors can be considered ductile. The ultimate resistance of composite beams with full or partial connection can be calculated using plastic methods.

The effect of the profiled steel sheet on the strength and deformation capacity of the HVB connectors is similar to that for welded studs. Consequently the same reduction formula can be applied to them.

In conclusion, it has been shown that the HVB connectors are a viable alternative to welded studs. Composite beams with HVB connectors behave in a similar fashion to beams connected by welded studs.

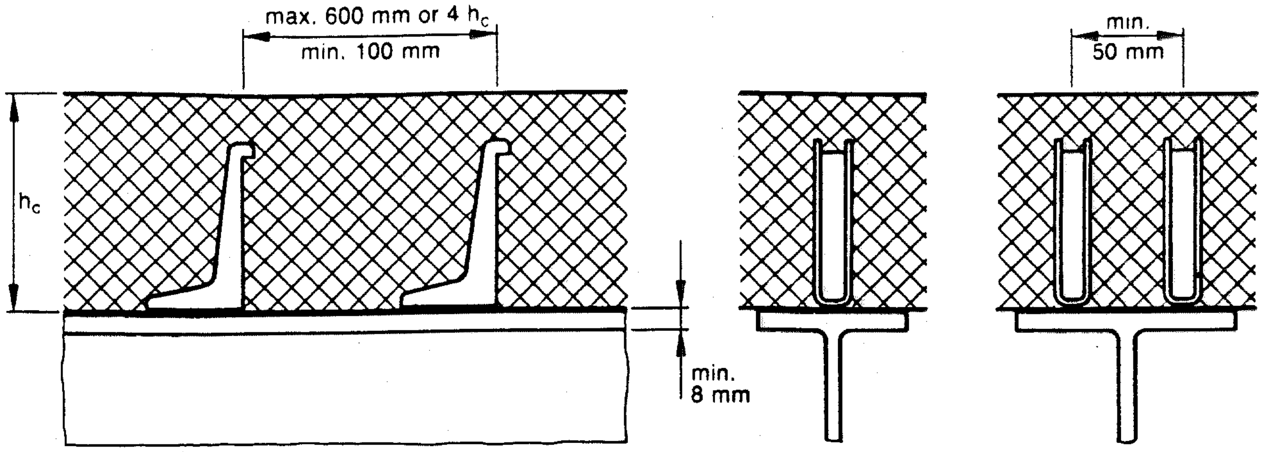


FIGURE 12 Detailing rules for a solid concrete slab

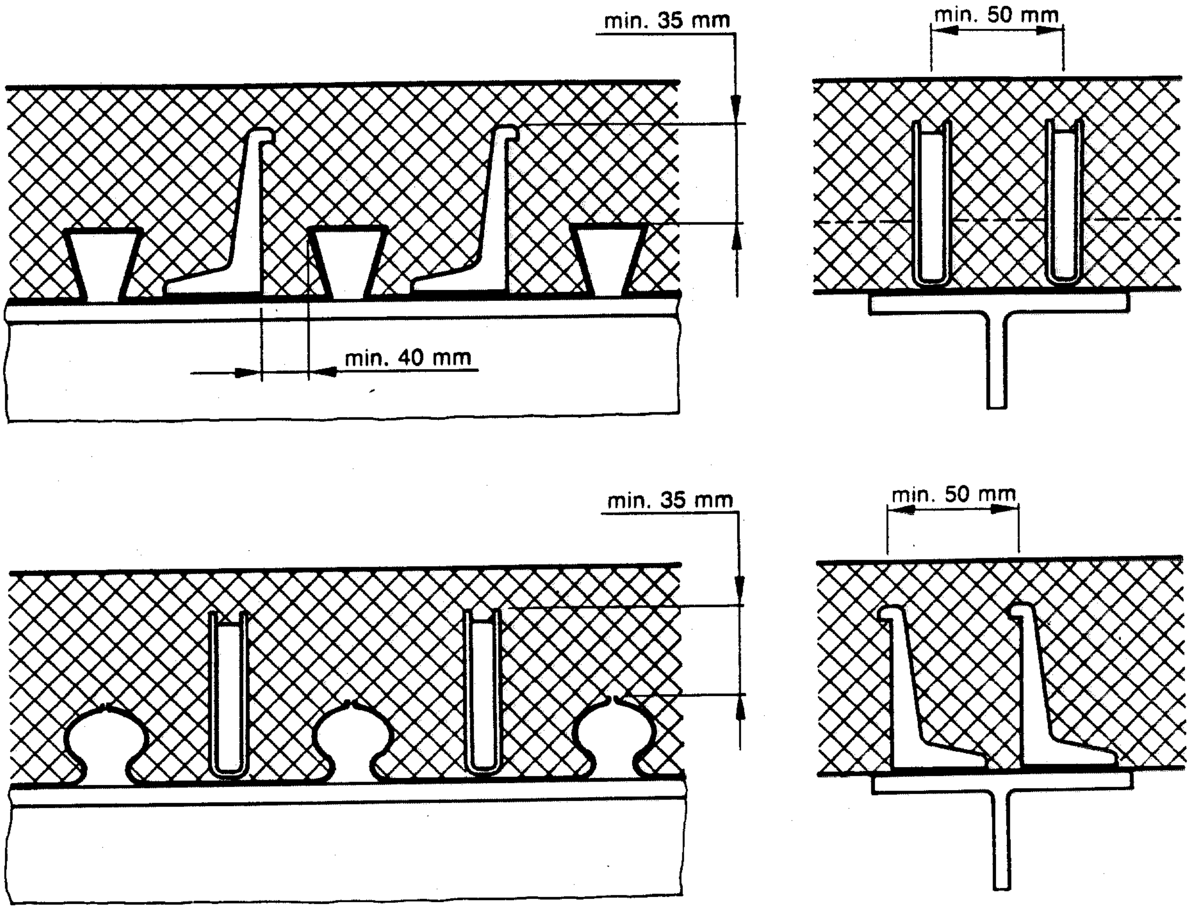


FIGURE 13 Detailing rules for a compact profiled sheet transverse to the beam

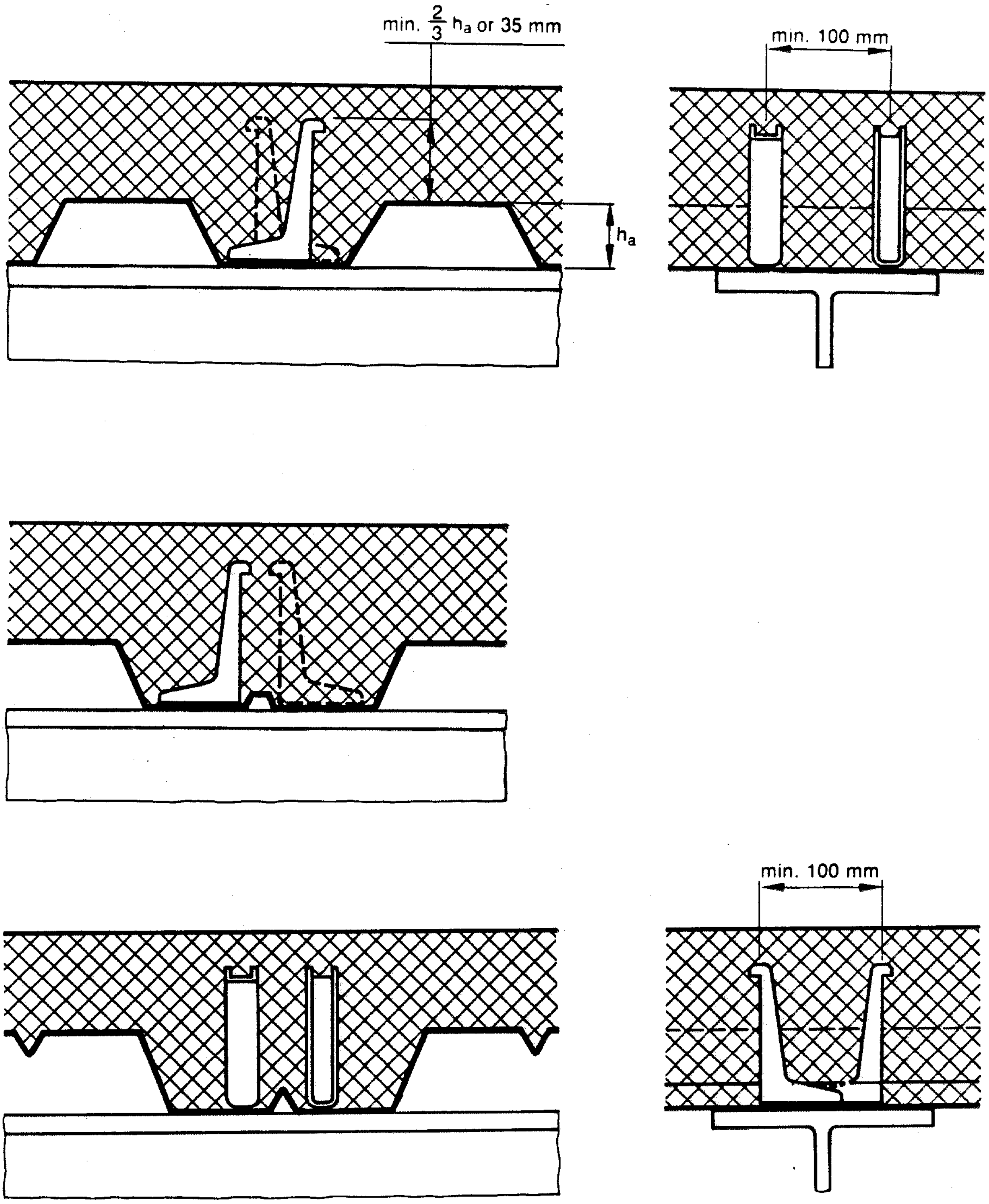


FIGURE 14 Detailing rules for a semi-compact profiled sheet transverse to the beam

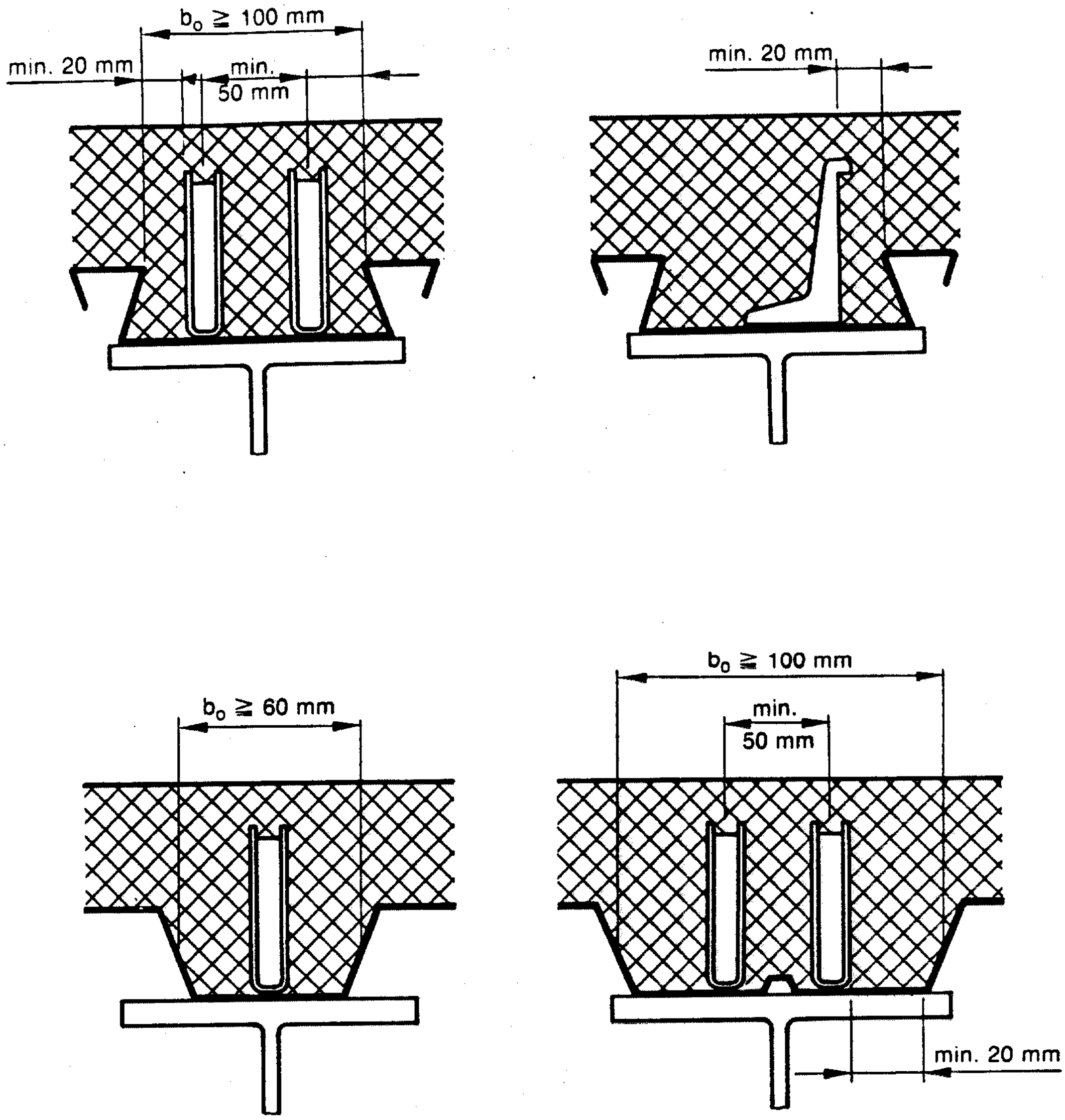


FIGURE 15 Detailing rules for a sheet parallel to the beam

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