

National Technical Approval

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Applicant:

Hoesch Bausysteme GmbH

Hammerstrasse 11

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Validity

from: 8 August 2016

to: 5 February 2021

Subject of approval:

Hoesch Additive Floor®

The subject of approval specified above is hereby given National Technical Approval. This National Technical Approval consists of 17 pages and 19 annexes.

This National Technical Approval replaces the National Technical Approval no. Z-26.1-44 of 5 February 2016. The subject received National Technical Approval for the first time on 7 January 2003.

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I GENERAL PROVISIONS

- 1 With the National Technical Approval, the usability and/or applicability of the subject of approval is proven in terms of the Landesbauordnungen [Federal States Construction Ordinances].
- 2 If requirements for the specialist knowledge and experience of those entrusted with the manufacture of construction products and construction types are made as per the relevant Federal State regulations corresponding to § 17 section 5 of the Musterbauordnung [Model Building Code] in the National Technical Approval, then it should be noted that this specialist knowledge and experience may also be documented through equivalent evidence of other member states of the European Union. This also applies to equivalent evidence submitted as part of the Agreement on the European Economic Area (EEA) or other bilateral agreements.
- 3 The National Technical Approval does not replace the legally prescribed approvals, agreements, and certifications for the execution of construction projects.
- 4 The National Technical Approval is issued irrespective of the rights of third parties, in particular of private property rights.
- 5 Manufacturers and distributors of the subject of approval must, irrespective of more extensive regulations in the "Special provisions", make available to the user and/or operator of the subject of approval copies of the National Technical Approval and refer to the fact that the National Technical Approval must be available at the location of use. Upon request, copies of the National Technical Approval must be made available to the authorities involved.
- 6 The National Technical Approval may be copied in its entirety only. Publication of extracts requires the agreement of the German Institute for Building Technology. Texts and drawings in promotional literature must not contradict the National Technical Approval. Translations of the National Technical Approval must contain the statement "This translation of the German original document has not been reviewed by the German Institute for Building Technology".
- 7 The National Technical Approval is issued on the basis that it is revocable. The provisions of the National Technical Approval may be retrospectively extended and modified, especially if new technical findings make this necessary.

II SPECIAL PROVISIONS

1 Subject of approval and area of applicability

The approved type is a load-bearing ceiling in accordance with Annex 1 or Annex 2 that is comprised of the TRP 200 steel trapezoidal profile panels of the Hoesch Roof System 2000 following the National Technical Approval no. Z-14.1-137 and an on-site produced reinforced concrete rib floor in accordance with DIN EN 1992-1-1¹.

In deviation from approval no. Z-14.1-137, the profile panels are suspended between steel girders and steel cleats with bearing support. The steel cleats are welded onto the steel girder upper flange and project laterally. In building condition, the profile panels are used as self-supporting formwork.

The approval applies for the use of the Hoesch Additive Floor® in car parks, as well as building construction and industrial construction and static and quasi-static load. In the limit conditions of load capacity and suitability for use, the floor can be idealised as a chain of uniaxially spanned, calculated jointed, bearing-supported, single-field panels. When using shaped sheet metal parts a formation as continuous girder is also possible.

For floor support spans ≤ 6.00 m and load capacities $q_k \leq 5.00$ kN/m² in accordance with DIN EN 1991-1-1², section 6.3, for the calculation the anisotropic load-bearing behaviour of the floor can be ignored, if the provisions of this approval are complied with. When using the floor in car parks, in addition section 3.4.2 must be complied with. For floor support spans > 6.00 m the anisotropic load-bearing behaviour of the floor must be complied with.

2 Provisions for the construction products

2.1 Characteristics and composition

2.1.1 Dimensions

2.1.1.1 Profile panels

The dimensions and dimensional tolerances of the profile panels must correspond to the information in Annexes 3 to 5 and to the information filed with the Deutsches Institut für Bautechnik.

For the limit dimensions of the nominal sheet thickness, the tolerances in accordance with DIN EN 10143³, Table 2 (normal limit dimensions), however for the lower limit dimension only the restricted limit dimension S applies.

2.1.1.2 Steel cleats

The dimensions and dimensional tolerances of the steel cleats must correspond to the information in Annexes 3 to 5 and to the information filed with the Deutsches Institut für Bautechnik.

2.1.1.3 Shaped sheet metal part for production of the continuity effect

The provisions in accordance with 2.1.1.1 apply.

¹ DIN EN 1992-1-1:2011-01

Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings General rules and rules for buildings in conjunction with DIN EN 1992-1-1/NA:2011-01

² DIN EN 1991-1-1:2010-12

Eurocode 1: Actions on structures – Part 1-1: General actions on structures – weighting, deadweight

³ DIN EN 10143:2006-09

Continuously hot-dip coated steel sheet and strip – Tolerances on dimensions and shape

2.1.2 Materials

2.1.2.1 Profile panels

For production of the profile panels a corrosion-protected sheet steel that is suitable for cold forming must be used.

The raw material must at least have the mechanical characteristics of type S350GD+Z steep in accordance with DIN EN 10346⁴.

These requirements must also be fulfilled by the finished component in the final use state.

2.1.2.2 Steel cleats

For the production of the steel cleats structural steel in accordance with DIN EN 1993-1-1⁵, Table 3.1 must be used.

2.1.2.3 Shaped sheet metal part for production of the continuity effect

The provisions in accordance with 2.1.2.1 apply.

2.1.3 Corrosion protection

The provisions in accordance with DIN EN 103 464 and DIN EN 1090-2⁶ and DIN 55634 apply⁷

As corrosion protection, at least a coating in accordance with requirement code Z275, ZA255 or AZ150 in DIN EN 10346⁴ must be provided.

2.2 Production and marking

2.2.1 Production

DIN EN 1090-2⁶ applies for the production of the profile panels. The in-house production control of the manufacturer must be certified in accordance with DIN EN 1090-1⁸.

2.2.2 Marking

In addition to the CE mark in accordance with EN 1090-1⁸, section ZA.3, a mark must be affixed that enables unique allocation to this national technical approval.

3 Provisions for design and calculation

3.1 General information

In the final state the profile panels and the reinforced concrete rib floor are additively load-bearing, i.e. a composite between profile panel and ribbed concrete floor is not claimed.

If nothing to the contrary is specified hereinafter, for the design detailing and calculation of the two components of the floor, DIN EN 1993-1-3⁹, DIN EN 1992-1-1¹ and DIN EN

⁴ DIN EN 10346:2015-10

Continuously hot-dip coated steel flat products for cold forming - Technical delivery conditions

⁵ DIN EN 1993-1-1:2010-12

Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings in conjunction with DIN EN 1993-1-1/NA:2010-12

⁶ DIN EN 1090-2:2011-10

Execution of steel structures and aluminium structures – Part 2: Technical requirements for steel structures

⁷ DIN 55634:2010-04

Paints, varnishes and coatings – Corrosion protection of supporting thin-walled building components made of steel

⁸ DIN EN 1090-1:2012-02

Execution of steel structures and aluminium structures - Part 1: Requirements for conformity assessment of structural components

⁹ DIN EN 1993-1-3:2010-12

Eurocode 3: Design of steel structures – Part 1-3: General rules – Additional rules for cold-formed, thin-walled components and sheet metals in conjunction with DIN EN 1993-1-3/NA:2010-12

1994-1-1¹⁰ apply. The arrangement of transverse ribs in accordance with DIN EN 1992-1-1¹, section 5.3.1 is not required.

The Hoesch Additive Floor[®] may also be used as a top flange of composite steel girders.

3.2 Design

3.2.1 Profile panels

The nominal sheet metal thickness must be 1.00 mm, 1.13 mm, 1.25 mm or 1.50 mm.

3.2.2 Concrete

The top concrete layer must correspond to the strength classes C20/25 to C50/60 in accordance with DIN EN 206-1 and ¹¹DIN 1045-2¹².

The thickness of the top flange plate of the ribbed floor must be at least 80 mm.

3.2.3 Bedding

The forming of the steel cleats and their fastening on the steel girders, as well as the support of the profile panels on the steel cleats must correspond to the information in Annexes 3 to 5. The profile panels must be fastened on each cleat with cartridge-fired pins that have National Technical Approval or European Technical Approval.

3.2.4 Bearing reinforcement of reinforcement steel

As design floor reinforcement against shrinkage cracks and for load distribution an orthogonal reinforcement network of at least 2.00 cm²/m must be placed in the top flange plate with due consideration of the concrete covering in accordance with DIN EN 1992-1 -11. This reinforcement must be taken into account for all verifications.

For a formation of the floor as a chain of single span girders, via inner girders usually as upper reinforcement must be arranged for delimitation of the crack widths in accordance with section 3.4.4.1. If the floor of a parking deck will not be directly driven on and the corrosion protection is otherwise ensured (e.g. through liner material, mastic asphalt), the smaller required concrete coverings on a upper reinforcement layer can be dispensed with, if the continuous reinforcement satisfies the requirements stipulated in section 3.4.4.1.

3.2.5 Bearing reinforcement of reinforcement steel

In each of the concrete ribs an underlying reinforcement bar with a minimum diameter of 8 mm must be inserted over the entire length of the profile panel. Its position in the cross section is shown in Annex 6.

On intermediate supports and end supports, suitable two-shear, inclined stirrups with incline under 45° must be inserted in the ends of the concrete ribs (see Annexes 7, 8, and 9). The diameter of the stirrups must be at least 6 mm. Additional required reinforcement (e.g. for absorbing the shoulder shear) must be separately verified.

At formation of the floor as continuous system, the continuous floor reinforcement must

¹⁰ DIN EN 1994-1-1:2010-12

Eurocode 4: Design of composite steel and concrete structures – Part 1-1: General rules and application rules for buildings in conjunction with DIN EN 1994-1-1/NA:2010-12

¹¹ DIN EN 206-1:2001-07

Concrete – Part 1: Specification, performance, production and conformity

¹² DIN EN 1045-2:2008-08

Concrete, reinforced and pre-stressed concrete structures – Part 2: Concrete – specification, performance, production and conformity – Application rules for DIN EN 206-1

be formed in accordance with static requirements. The upper reinforcement layer for delimitation of crack widths in accordance with section 3.4.4.1 can be dispensed with, if the continuous floor reinforcement meets the requirements specified in section 3.4.4.2.

3.2.6 Bracing

For the transmission of horizontal forces and for the horizontal bracing of storey constructions only the top flange plate can be used. In this regard, simultaneously stresses acting in the floor plane and transverse to the floor plane must be overlaid.

The further transmission of the horizontal forces into the subconstruction or vertical composites or slabs must be verified.

3.2.7 Edging

The edge of the floor parallel to the rib must be formed as specified in Annex 10.

3.2.8 Parking decks

Parking decks must be executed with an incline of at least 1.5%. Adequate drainage must be ensured for the entire floor surface.

3.3 Calculation of the profile panels in building condition

3.3.1 Load assumptions

In addition to the inherent weight of the profile panels and of the unset concrete with reinforcement, for the concreting and other erection workloads that must be taken into account in accordance with DIN EN 1994-1 -1¹⁰, section 9.3.2 in conjunction with DIN EN 1991-1-6¹³, section 4.11.2 must be assumed.

3.3.2 Structural safety verification

For the structural safety verification of the profile panels the verifications in accordance with DIN EN 1993-1 -3⁹ apply.

The stress factors and calculation parameters for the profile panel are provided in Annex 12. A transverse seam of the panels is not permitted.

If temporary intermediate supports are required in the building condition, then these must be formed and calculated in accordance with the provisions for intermediate supports specified in National Technical Approval no. Z-14.1-137.

For verification of the absorption of transverse forces the shearing of the sheet in the area of the profile panel bearing arrangement on the cleat is authoritative. The appropriate calculation values $A_{K,Rd}$ of the absorbable transverse force per cleat are provided in Table 1. At formation of the fastening of the steel cleat on the steel girder in accordance with Annexes 3, 4, or 5, a separate verification of the fastening can be dispensed with.

Table 1: Stress factor of the profile panel bearing arrangement on a steel cleat

t_{nom} [mm]	1.00	1.13	1.25	1.50
$A_{K,Rd}$ [kN]	7.9	9.3	10.8	14.1

The cartridge-fired pin with which the profile panel is fastened on the steel clean must be verified for horizontal shear-off for a force $F_{Qd} = 0.25 A_{K,Ed}$, where $A_{K,Ed}$ is the calculation value of the bearing force attributed to one steel cleat.

Any torsion stresses of the steel girders during concreting due to one-sided load of the unset concrete must be taken into account.

¹³ DIN EN 1991-1-6:2010-12

Eurocode 1: Actions on structures – Part 1-6: General effects, effects during construction execution in conjunction with DIN EN 1991-1-6/NA:2010-12

3.4 Calculation of the floor in the end state

3.4.1 Calculation bases

The verification concept specified in DIN EN 1990¹⁴ applies.

The calculation model in Annex 11 is the basis for verification of structural safety. It is characterised by the fact that

- The bending moment $M_{Ed,max}$ of the floor is additively absorbed by the profile panel and by the reinforced concrete rib floor and
- The transverse force $V_{Ed,max}$ on the support of the floor is likewise additively absorbed by the profile panel and by the reinforced concrete rib floor, whereby the assumption is made that the profiled metal sheet are not supported in the building condition.

In this regard, the proportion $q_{c,Ed}$ of the section load of the reinforced concrete rib is transmitted over the entire support width L , while the proportion $q_{PT,Ed}$ is dissipated by the profiled metal sheet over the support width L_{PT} . Hereinafter, the spacing between the bending points of the rib reinforcement is designated as L_c . See Annex 11 The mathematical support width L_{PT} of the profile panel arises from the spacing of the cleat supports and the support width L equals the spacing of the supporting composite girders.

3.4.2 Load assumptions

For vertical service loads, for concentrated point loads, or for linear loads that are greater than those cited below, special measures are required that are not the subject of this approval.

For traffic and parking areas for light vehicles (total load ≤ 30 kN) that are verified with an area load/service load $q_k \leq 5.0$ kN/m², verifications with axle load $2 \cdot Q_k$ or wheel load Q_k in accordance with DIN EN 1991-1-1², section 6.3.3 can be dispensed with, if a stirrup reinforcement in accordance with Annexes 7 to 9 is arranged in the support areas.

Unloaded lightweight partition walls may be taken into account through an addition Δq_k to the service load in accordance with DIN EN 1991-1-1², section 6.3.1.2 (8). If a more precise verification of an adequate transverse distribution is not provided, then due to the anisotropic formation of the floor, this addition must be increased by one third for the calculation of the floor.

3.4.3 Verifications for limit states of the load capacity

3.4.3.1 Verification of the absorbable bending moments

The absorbable moment M_{Rd} arises from the sum of the bending stress factors of the profile panel ($M_{PT,Rd}$) and of the reinforced concrete rib floor (M_{cRd}):

$$M_{Rd} = (M_{PT,Rd}) + (M_{cRd}) \quad (1)$$

The bending load of the profile panel is

$$M_{PT,Rd} = M_{PT,Rk} / \gamma_{M1} \quad (2)$$

with $\gamma_{M1} = 1.1$ and $M_{PT,Rk}$ in accordance with Annex 12.

The bending stress factor of the reinforced concrete rib floor $M_{c,Rd}$, must be determined in accordance with DIN EN 1992-1-1¹, section 6.1. In this regard the cross-section area of the reinforcement per rib must not be calculated as more than 2.6 cm², even if, for

¹⁴ DIN EN 1990:2010-12

Eurocode: Fundamentals of support structure planning in conjunction with DIN EN 1990/NA:2010-12

example, more reinforcement is inserted in the ribs for fire safety reasons (see section 3.4.3.6.2). The moment load capacity $M_{c,Rd}$ at positive moment stress (pressure zone in the top concrete layer) may also be approximately determined in three-dimensions in agreement with DIN EN 1994-1 -1¹⁰.

The load components $q_{PT,Ed}$ and $q_{c,Ed}$ of the calculated load q_{Ed} applied on the profiled sheet metal and the reinforced concrete rib floor arise at equal section loads for:

$$q_{PT,Ed} = q_{Ed} \frac{M_{PT,Rd}(L/L_{PT})^2}{M_{c,Rd} + M_{PT,Rd}(L/L_{PT})^2} \geq q_{G,Ed} \quad (3)$$

$$q_{PT,Ed} = q_{Ed} - q_{PT,Ed} \quad (4)$$

In this regard $q_{G,Ed}$ is the calculated value of the action arising from the inherent weight of the profile panel and the inherent weight of the concrete.

3.4.3.2 Verification of the absorbable transverse forces

The calculated value of the transverse tensile capacity of the floor in the support area is comprised of the transverse tensile capacity of the profile panel and of the reinforced concrete rib floor, including the inclined stirrups present on the supports. Due to the different deformation behaviour the load capacity of the individual components cannot be fully exploited. In Table 2 this is taken into account with the factors k_c and k_s .

For the profiled metal sheet the following must be verified:

$$V_{PT,Ed} / V_{PT,Rd} \leq 1.0 \quad (5)$$

In this regard $V_{PT,Ed}$ is the transverse force of the profiled sheet metal on the mathematical stirrup support resulting from $q_{PT,Ed}$ and $V_{PT,Rd} = V_{Rd,K}$ is the calculation value of the transverse tensile capacity component of the trapezoidal sheet in accordance with Table 6. At formation of the fastening of the steel cleat on the steel girder in accordance with Annex 3, 4, or 5, a separate verification of the fastening can be dispensed with.

For the reinforced concrete rib floor the verification of adequate load capacity is provided if on the mathematical support the condition

$$V_{c,Ed} / V_{c,Rd} \leq 1.0 \quad (6)$$

is fulfilled. $V_{c,Ed}$ is the transverse force resulting from $q_{c,Ed}$ on the mathematical support of the reinforced concrete rib floor (see Annex 11). The calculated value of the transverse tensile capacity for the influence area of a rib with a reference width of $b = 750$ mm is:

$$V_{c,Rd} = k_c \cdot V_{Rd,c,min} + k_s \cdot V_{Rd,s} \quad (7)$$

In this regard:

k_c, k_s are factors for consideration of the different deformation behaviour of the individual components in accordance with Table 2

$V_{Rd,c,min}$ minimum value of the transverse tensile capacity in accordance with Table 3

$V_{Rd,s}$ calculated value of the transverse tensile capacity component of the inclined stirrups in the reinforced concrete ribs in accordance with Table 4 for the intermediate support and in accordance with Table 5 for the end support with arrangement of the stirrups in accordance with Annexes 7 to 9

Table 2: Factors k for consideration of the load-bearing proportion of the individual components

Component	Factor	Edge supports	Intermediate support in accordance with Annex 3	Intermediate support in accordance with Annex 4
Top concrete layer	k_c	0.50	0.60	0.30
Suspended reinforcement	k_s	0.85	1.00	1.00

Table 3: Calculated value $V_{Rd,c,min}$ of the transverse tensile capacity component of the reinforced concrete rib in [kN] based on the rib spacing $b = 750$ mm

Transverse force load capacity [kN]	Top concrete layer thickness h_c [cm]	Concrete strength class					
		C20/25	C25/30	C30/37	C35/45	C40/50	C45/55
$V_{Rd,c}$	8 cm	10.1	11.3	12.3	13.3	14.2	15.1
	9 cm	10.3	11.6	12.7	13.7	14.6	15.5
	10 cm	10.6	11.8	13.0	14.0	15.0	15.9

Table 4: Calculated value $V_{Rd,s}$ of the transverse tensile capacity component of the shear reinforcement of a rib in [kN] on the intermediate support for $b = 750$ mm

Transverse load capacity [kN]	Reinforcement diameter ϕ [mm]				
	6	7	8	9	10
$V_{Rd,s}$	17.4	23.6	31.0	39.1	48.3

Table 5: Calculated value $V_{Rd,s}$ of the transverse tensile capacity component of the shear reinforcement of a rib (reference width = 750 mm) in [kN] on the end support

Transverse load capacity [kN]	Concrete strength class	Reinforcement diameter ϕ [mm]				
		6	7	8	9	10
$V_{Rd,s}$	C20/25	8.6	10.1	11.5	13.0	14.4
	C25/30	10.0	11.7	13.4	15.0	16.7
	C30/37	11.3	13.2	15.1	17.0	18.9
	C35/45	12.5	14.6	16.7	18.8	20.9
	C40/50	13.0	16.0	18.3	20.6	22.8
	C45/55	13.0	17.3	19.8	22.2	24.7

Table 6: Calculated value $V_{Rd,K}$ of the transverse tensile capacity component of a rib of the trapezoidal sheet in [kN] (load capacity per stirrup pair)

Transverse load capacity [kN]	Nominal thickness of the trapezoidal sheets t_{nom} [mm]			
	1.00	1.13	1.25	1.50
$V_{Rd,K}$	13.5	16.2	19.0	25.2

In addition, the transverse tensile capacity of the reinforced concrete rib floor outside of the support area must also be verified. The calculated value of transverse force $V_{c,rib,Ed}$ on the reinforced concrete rib in accordance with Annex 11 acting outside of the support area must not exceed the transverse tensile capacity $V_{Rd,c}$ in accordance with DIN EN 1992-1 -1¹, section 6.2.2(1). The rib width at the height of the longitudinal reinforcement must be assumed as the smallest cross-section diameter b_w (see Annex 6).

3.4.3.3 Verification of the anchoring of the flexural tension reinforcement in the rib on the support

The overlap length of the suspended reinforcement with the rib length reinforcement must be verified in accordance with DIN EN 1992-1 -1¹, section 8.7.3 for the full tensile force of the suspended reinforcement.

3.4.3.4 Calculation of the joists as composite steel girders

3.4.3.4.1 Use of the cleats as systematic composite material

The steel cleats fastened on the steel girder top flange must be classified as a non-ductile composite in accordance with DIN EN 1994-1-1¹⁰. For use of the steel cleats as systematic composite material the verification of the composite steel joist of the Hoesch Additive Floor[®] must be provided with the aid of the calculation diagrams in Annex 14. depending on the utilisation factor from bending moment stress η_M , the maximum longitudinal shear force $V_{L,Ed,max}$ within the girder length is provided in the diagram, or alternatively it can be determined via the equations (9) to (12).

$$\eta_M = \frac{M_{Ed}}{M_{pl,Rd}} \quad (8)$$

In this regard:

M_{Ed} is the calculated value of the bending moment stress from outer loads

$M_{pl,Rd}$ three-dimensional bending moment capacity of the composite cross-section in accordance with DIN EN 1994-1 -1¹⁰

The maximum longitudinal shear force arises at a production of the composite girder without inherent weight composite

$$V_{L,Ed,max} = V_{L,Ed,A} \quad \text{for } \eta_M \leq 0.75 \quad (9)$$

$$V_{L,Ed,max} = V_{L,Ed,A} \cdot \left(1 + 7,5 \left(\frac{M_{Ed}}{M_{pl,Rd}} - 0,75 \right) \right) \quad \text{for } 0.75 < \eta_M \leq 0.95 \quad (10)$$

and at production of the composite girder with inherent weight composite

$$V_{L,Ed,max} = V_{L,Ed,A} \quad \text{for } \eta_M \leq 0.95 \quad (11)$$

$$V_{L,Ed,max} = V_{L,Ed,A} \cdot \left(1 + 3,0 \left(\frac{M_{Ed}}{M_{pl,Rd}} - 0,95 \right) \right) \quad \text{for } 0.95 < \eta_M \leq 1.00 \quad (12)$$

In this regard $V_{L,Ed,A}$ is the mathematical longitudinal shear force on the support according to equation (13). For girders with an inherent weight composite the longitudinal shear force on the support may be determined with due consideration of creep and shrinking, as well as load history.

$$V_{L,Ed,max} = \frac{A_{c,0} \cdot z_{ic,0}}{I_{i,0}} V_{c,Ed} [\text{kN/m}] \quad (13)$$

with

$A_{c,0}$	based on the modulus of elasticity of the structural steel, reduced concrete area of the composite cross-section at time $t = 0$
$z_{ic,0}$	distance of the ideal centre of gravity of the composite cross-section to the centre of gravity of the concrete area at time $t = 0$
$I_{i,0}$	ideal geometrical moment of inertia of the composite cross section at time $t = 0$
$V_{c,Ed}$	component of the transverse force on the support acting on the composite cross section

It must be verified that the maximum longitudinal shear force in the composite joint does not exceed the longitudinal load capacity of the cleats.

$$V_{L,Ed,max} \leq V_{L,Rd} \quad (14)$$

For the longitudinal load capacity of the cleats, at compliance with the design boundary conditions, in accordance with Annex 15, shear-off at the height of the upper edge of the cleat is authoritative. Thus a verification of the load capacity of the pure cleat cross-section, of the weld seams for fastening the steel girder flange, as well as the partial area pressure of the concrete directly in front of the cleat, is not required.

The longitudinal shear force load capacity of the cleats at an existing distance of ribs of 750 mm relative to each other and arrangement of 2 cleats for rib arises with equation (15) from the absorbable force per cleat

$$V_{L,Rd} = \frac{n}{e} \cdot P_{Rd} = \frac{P_{Rd}}{0,375} \quad (15)$$

In this regard:

n	= 2, number of cleats per suspension point
e	= 0.75 m, average spacing of the cleat pairs in the girder longitudinal direction
P_{Rd}	calculated value of the longitudinal shear force capacity of a cleat in accordance with equation (16)
$P_{Rd} = P_{Rd,c} + P_{Rd,s}$	(16)
$P_{Rd,c}$	component of the tensile strength of the concrete on the longitudinal shear force load capacity of a cleat in accordance with Table 7
$P_{Rd,s}$	component of this shear area intersecting reinforcement (connector) on the longitudinal force load capacity of a cleat in accordance with Table 8

Table 7: Component of the calculated value of the longitudinal shear force load capacity arising from the tensile strength of the concrete $P_{Rd,c}$ [kN] based on a cleat

Flange width b_f [mm]	Concrete strength class					
	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55
200	103.2	119.7	135.2	149.8	163.7	177.1
250	112.8	130.9	147.8	163.8	179.1	193.7
300	122.5	142.1	160.5	177.9	194.4	210.3

Table 8: Component of the calculated value of the longitudinal shear force load capacity arising from the shear force intersecting reinforcement (connector) $P_{Rd,s}$ [kN] based on a cleat

Diameter $\phi_{s,st}$ of the connectors in the cleats [mm]	6	8	10
$P_{Rd,s}$ [kN]	20.9	37.2	58.1

At this point as an alternative to an arrangement of connectors for anchoring of the shear band, the use of tension anchors is also possible. As tension anchors stud shear connectors in accordance with DIN EN ISO 13918¹⁵ with small shaft diameters $d_z \leq 13$ mm may be used. For a systematic arrangement of a tension anchor centered between 2 steel cleats the load capacity components in accordance with Table 9 result. Load capacity component $P_{Rd,s}$ in equation (16) in this case must be replaced with the component $P_{Rd,z}$.

Table 9: Component of the calculated value of the longitudinal shear force load capacity arising from the shear force intersecting tension anchors $P_{Rd,z}$ [kN] based on a cleat

Diameter d_z of the tension anchors [mm]	10	13
$P_{Rd,z}$ [kN]	7.0	11.8

A utilization of the com positive girder manufactured without inherent weight composites in plastic condition greater than $0.95 \cdot M_{pl}$ is not permitted.

3.4.3.4.2 Use of ductal composite materials for transmission of longitudinal shear forces

If for systematic transmission of the longitudinal shear forces, ductile composite material in accordance with DIN EN 1994-1-1¹⁰ is used, a combined logic to hear force load carrying action with the steel cleats must be prevented through elastic cushioning of the vertical contact areas of the steel cleats. The execution of the elastic cushioning must be undertaken in accordance with Annex 17. In this regard the elastic material must show a minimum value of the form ability of 4 mm at a pressure of 2 N/mm². Ductile stud shear connectors in accordance with DIN EN ISO 13918¹⁵ that must be dimensioned in accordance with DIN EN 1994-1 -1¹⁰ may be used as ductile composite material.

For the lift-off protection of the floor as top flange of a composite girder, an application of section 6.6.5.1 of DIN EN 1994-1 -1¹⁰ can be dispensed with, if the top concrete layer thickness under the trapezoidal sheet is not greater than 12 cm.

3.4.3.5 Verification of the floor as top flange of composite steel girders

If the floor is used as top flange for composite steel girders, then the connection of the top flange plate must be verified. For the calculated value of the acting longitudinal shear force $V_{L,Ed}$ the calculated value that must be assessed should be used for verification of the composite material.

The longitudinal shear force load capacity $V_{L,Rd}$ in the panel section must be determined in accordance with DIN EN 1992-1 -1¹, section 6.2.4 or DIN EN 1994-1-1¹⁰, section 6.6.6.2.

The anchoring of the transverse reinforcement must be separately verified in particular for edge girders.

¹⁵ DIN EN ISO 13918:2008-10

Welding – Studs and ceramic ferrules for arc stud welding (ISO 13918:2008)

3.4.3.6 Loadability under fire exposure

3.4.3.6.1 General information

Verification of the classification in a fire resistance class in accordance with DIN 4102-2¹⁶ or DIN EN 1992-1-2¹⁷ must be provided with the recordable cutting sizes in accordance with section 3.4.3.6.2 and section 3.4.3.6.3.

As partial safety factor in the event of fire $\gamma_{M,fi} = 1.0$ must be used.

The classification only applies if the supporting components belong to at least the same the fire resistance class as the floor.

3.4.3.6.2 Absorbable bending moment in the event of fire

The calculated value of the fire reduced bending load capacity of the reinforced concrete rib floor $M_{c,Rd,fi}$ per rib is

$$M_{c,Rd,fi} = \frac{1}{\gamma_{M,fi}} A_s k_1 f_{sk} \left[d - 0,5 \frac{A_s k_1 f_{sk}}{0,85 f_{ck} b} \right] \quad (17)$$

Here the following mean:

$\gamma_{M,fi}$ partial safety factor $\gamma_{M,fi} = 1.0$

A_s cross-section area of the reinforcing steel per rib with $A_s \leq 5.0 \text{ cm}^2$,

f_{sk} characteristic value of the tensile yield strength of the reinforcing steel,

f_{ck} characteristic value of the cylindrical compressive strength of the concrete

b Rib spacing = 750 mm,

d Usable height of the reinforced concrete rib,

k_1 factor in accordance with Table 10 for detecting the fire reduction of the tensile yield point of the reinforcing steel.

Table 10: Fire reduction factors k_1

Fire resistance class	Axial spacing u [mm] of the rib steel from the lower edge of the rib ¹⁾			
	40	50	60	70
F 30 / REI 30	1.00	1.00	1.00	1.00
F 60 / REI 60	0.95	1.00	1.00	1.00
F 90 / REI 90	0.45	0.60	0.70	0.80

¹⁾ For intermediate values of u linear interpolation is permitted.

3.4.3.6.3 Maximum absorbable transverse force in the event of fire

The profile panel bearing arrangement on the steel cleat must not be included in the calculation for the dissipation of transverse forces under fire exposure. The total transverse force of the floor must be dissipated via the reinforced concrete ribs. For this, a fire safety suspended reinforcement in accordance with capital annexes 18 and 19 must be arranged. The design suspension reinforcement in accordance with Annexes 7, 8 and 9 can be omitted in this case.

The calculated value of the fire reduced transverse force capacity of the suspended reinforcement $V_{c,Rd,fi}$ per rib is equal to the lower of the two following values

¹⁶ DIN 4102-2:1977-09

Fire Behaviour of Building Materials and Building Components – Part 2; Building Components; Definitions, Requirements and Tests.

¹⁷ DIN EN 1992-1-2:2010-12

Eurocode 2: Design and construction of reinforced and pre-stressed concrete loadbearing structures – Part 1-2: General rules – structural fire design in conjunction with DIN EN 1992-1-2/NA:2010-12

$$V_{c,Rd,fi} = \frac{1}{\gamma_{M,fi}} A_{s,H} k_2 f_{sk} 0,85 h_c / a \quad (18)$$

$$V_{c,Rd,fi} = \frac{1}{\gamma_{M,fi}} A_{s,V} k_3 f_{sk} \quad (19)$$

Here the following mean:

$\gamma_{M,fi}$ Partial safety factor $\gamma_{M,fi} = 1.0$

$A_{s,H}$ cross-section area of the horizontal suspended reinforcement in accordance with Annexes 18 and 19,

$A_{s,V}$ cross-section area of the vertical suspended reinforcement in accordance with Annexes 18 and 19,

f_{sk} characteristic value of the tensile yield strength of the reinforcing steel,

h_c top flange plate thickness,

a distance of the centroidal axis of the vertical suspended reinforcement from the system line of the floor girder (see Annexes 18 and 19),

k_2, k_3 factors in accordance with Table 11 for identifying the fire reduction of the tensile yield point of the reinforcing steel.

Table 11: Fire reduction factors k_2 and k_3

Fire resistance class	k_2	k_3
F 30 / REI 30	1.00	1.00
F 60 / REI 60	0.80	0.60
F 90 / REI 90	0.50	0.30

3.4.4 Verifications for limit states of usability

3.4.4.1 Delimitation of the crack width when forming the floor as a chain of single span girders

If a more precise verification is not provided, then in the case of predominant bending constraint a minimum reinforcement must be arranged via inner girders. The minimum reinforcement for crack moment M_R must be determined in accordance with equation (20) with due consideration of the limit stresses specified in Table 12. The limit value w_{max} for the mathematical crack width w_k must be determined depending on the exposure class in accordance with DIN EN 1992-1-1¹, Table 7 1.N.

$$M_R = k f_{ct,eff} \eta h_c^2 / 6 \quad (20)$$

In this regard:

$f_{ct,eff}$ effective concrete tensile strength for which the mean value of the centric tensile strength f_{ctm} in accordance with DIN EN 1992-1-1¹, Table 3.1 may be applied, however at least 3.0 N/mm²,

k coefficient for consideration of nonlinear distributed inherent stresses, that can be assumed with $k = 0.8$,

h_c top concrete layer thickness.

$$\eta = 1 + 0,18 / \sqrt{h_c} \text{ with } h_c \text{ in [m]}$$

In the case of predominant centric constraint a continuous minimum reinforcement is required that must be determined for the normal crack force N_R in accordance with equation (21) we do consideration of the limit stresses specified in Table 12, if it is not proven that the constraint force taking the crack formation into account is less than the normal crack force according to equation (21).

$$N_R = k f_{ct,eff} h_c \quad (21)$$

For k , $f_{ct,eff}$ and h_c the explanations for equation (20) apply.

Limit stress in accordance with Table 12 must be modified for components with predominant bending constraint and a concrete covering c_{nom} greater than 30 mm in accordance with equation (22).

$$\sigma_s = \sigma_{s,Tabelle} \cdot \frac{c_0 + d_s/2}{c_{nom} + d_s/2} \quad (22)$$

In this regard:

c_0 the reference value of the concrete covering ($c_0 = 30$ mm), that is the basis for the limit values specified in Table 12 for the stress in the reinforcing steel,

d_s of the used bar diameter,

c_{nom} the required concrete covering in accordance with DIN EN 1992-1-1¹.

Table 12: Limit values of the stress σ_s [N/mm²] for reinforcing steel in accordance with DIN EN 10080¹⁸

Crack width w_{max} [mm]	$d_s = 10$ mm	$d_s = 8$ mm	$d_s = 6$ mm	$d_s = 5$ mm
0.4	285	320	370	400
0.3	245	275	320	350
0.25	225	250	290	320
0.2	200	225	260	285

The concrete covering in accordance with DIN EN 1992-1-1¹, section 4.4.1 may be reduced, if instead of reinforcing steel in accordance with DIN EN 10080¹⁸ reinforcing steel made of steel B500B NR with National Technical Approval of at least corrosion resistance class III is used (classification accordance with the National Technical Approval Z-30.3-6). The concrete covering c_{nom} in this case must be at least 20 mm. At compliance with the appropriate boundary conditions, concrete covering in accordance with DIN EN 1992-1-1¹, section 4.4.1 can be reduced to 15 mm. The minimum reinforcement must be determined in accordance with equation (20) or (21), where the limit values of the stresses σ_s for stainless steel in accordance with Table 13 must be used. The stress in accordance with Table 13 must be modified at predominant bending constraint for a concrete covering c_{nom} greater than 15 mm in accordance with equation (22) where $c_0 = 15$ mm must be used.

Table 13: Limit values of the stress σ_s for stainless steel [N/mm²]

Crack width w_{max} [mm]	$d_s = 8$ mm	$d_s = 6$ mm	$d_s = 5$ mm
0.25	320	370	400

With predominant bending constraint the reinforcement on both sides must protrude at least 25 cm beyond the edges of the flange.

3.4.4.2

Delimitation of the crack width when forming the floor as a chain of continuous girders

For the alternative execution variant of the support area in accordance with Annex 4 the minimum reinforcement in the limit state of usability occurs with equation (23) depending on the constraint force.

¹⁸ DIN EN 10080:2005-08

Steel for the reinforcement of concrete – Weldable reinforcing steel

$$a_{s,min} = \frac{k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct}}{\sigma_s} \quad (23)$$

In this regard:

$$k = 0.8$$

$$f_{ct,eff} = f_{ctm} \geq 3.0 \text{ N/mm}^2$$

A_{ct} area of the continuous concrete slab

$$k_c = 1.0 \text{ at centric constraint}$$

$$k_c = 0.5 \text{ at pure bending constraint}$$

$$k_c = 0.77 \text{ at centric constraint}$$

σ_s Stress in the reinforcing steel in accordance with Table 12 at centric constraint

σ_s Stress in reinforcing steel in accordance with DIN EN 1992-1-1/NA¹, Table 7.2DE at pure bending constraint and combined stress

In addition it must be verified that the existing reinforcement diameter does not exceed the modified limit diameter of the reinforcement in accordance with equation (24). In this regard this section sizes for determination of the stress of the reinforcement must be determined with linear-elastic calculation procedures without moment overlay. The pressure zone height x in accordance with Annex 20 determination of the inner lever arm may be calculated simplified three dimensionally.

$$d_s = d_s^* \cdot \frac{\sigma_s \cdot A_s}{4(h-d) \cdot b \cdot f_{ct,0}} \geq d_s^* \cdot \frac{f_{ct,eff}}{f_{ct,0}} \quad (24)$$

In this regard:

d_s^* the limit diameter of the reinforcement in accordance with DIN EN 1992-1-1/NA¹, Table 7.2DE,

σ_s distress in the reinforcement arising from the quasi-continuous load combination,

A_s the area of the reinforcement in the tension zone,

h the total height of the cross-section,

d the static service height up to the center of gravity of the reinforcement,

b the width of the cross-section in the tension zone,

$f_{ct,0}$ = 2,9 N/mm², reference value of the concrete tensile strength,

$f_{ct,eff}$ the mean value of the effective tensile strength of the concrete at the time the cracks occur; if t is $t \geq 28 \text{ d}$, then $f_{ct,eff} = f_{ctm} \geq 3.0 \text{ N/mm}^2$ applies

3.4.4.3 Supplemental instructions for crack width delimitation

If the floor is simultaneously top flange of a composite girder (see section 3.4.3.4), then the resulting total reinforcement must be determined by the following equations (25) and (26). In this regard the larger value is authoritative.

$$erf a_s = a_{s,min} + 0.5 a_{s,T} \quad (25)$$

$$erf a_s = a_{s,T} \quad (26)$$

In this regard $a_{s,min}$ is the required minimum reinforcement in accordance with equation (20), (21) or (23) and $a_{s,T}$ is the required shoulder shear reinforcement in accordance with section 3.4.3.4.

For parking decks with predominant bending constraint, the minimum reinforcement $a_{s,min}$ of nonrusting steel may not be taken into account.

For parking decks that are directly driven on with predominant bending constraint crack formation must be assumed. For the most part this is probable in the area above floor girders

For parking decks that are directly driven on with predominant centric constraint crack formation on the entire floor surface must be assumed.

For assurance of the durability, in particular in the area of cracks, the provisions in DIN EN 1992-1-1¹, as well as in booklet 600:2012 and booklet 526:2010 issued by the DAfStb, must be complied with. Moreover, the "DAfStb Guideline for Protection and Repair of Concrete Components" (October 2001), for service protection systems DIN V 18026¹⁹ and for crack fillers DIN V 18028²⁰ must be complied with.

3.4.4.4 Delimitation of the deflection

For delimitation of deflection the rules in accordance with DIN EN 1992-1-1¹, section 7.4 may be applied.

4 Provisions for execution

For the concrete tasks the rules in accordance with DIN EN 13670²¹/DIN 1045-3²² must be complied with.

Depending on the requirements that are specified for the construction – in coordination with the structure planner and the governmental approval authority – for execution of the welding tasks on the steel cleats the provisions for execution classes EXC2 or EXC3 in accordance with DIN EN 1090-2⁶ apply.

Each profile panel, after installation with cartridge-fired pins, must be fastened on the cleats in accordance with Annexes 3, 4 and 5.

The profile panels must be fastened in the longitudinal seems and on the longitudinal edge with connecting elements with National Technical Approval at a spacing of maximum 666 mm. If the profile panels are used as a shear area then the number and arrangement of the connecting elements must be statically verified.

If in the building condition the profile panels are taken into account for the bracing of bearing structures, they must only be installed by structural steel specialist under the supervision of a specialized engineer. An acceptance protocol must be drawn up in this regard and must be confirmed by the responsible specialized engineer or construction manager. The acceptance protocol must be placed together with the technical building documents and submitted to the building supervisory authorities on request.

Low-shrinkage concrete with a low water-cement ratio must be used to the extent possible.

For section by section concreting it must be ensured that due to different deformations of the floor girders no significant constraints in the floor sections that are in the hardening phase occur.

It must be ensured that concrete suspension elements, whose weight exceeds the

¹⁹ DIN V 18026:2006-06 preliminary surface protection systems for concrete made of products in accordance with DIN EN 1504-2:2005-01

²⁰ DIN V 18028_2006-06 preliminary standard, crack fillers in accordance with DIN EN 1504-5:2005-03 with special properties

²¹ DIN EN 13670:2011-03 Execution of concrete structures

²² DIN EN 1045-3:2012-03 Concrete, reinforced and pre-stressed concrete structures – Part 3: Construction – application rules for DIN EN 13670

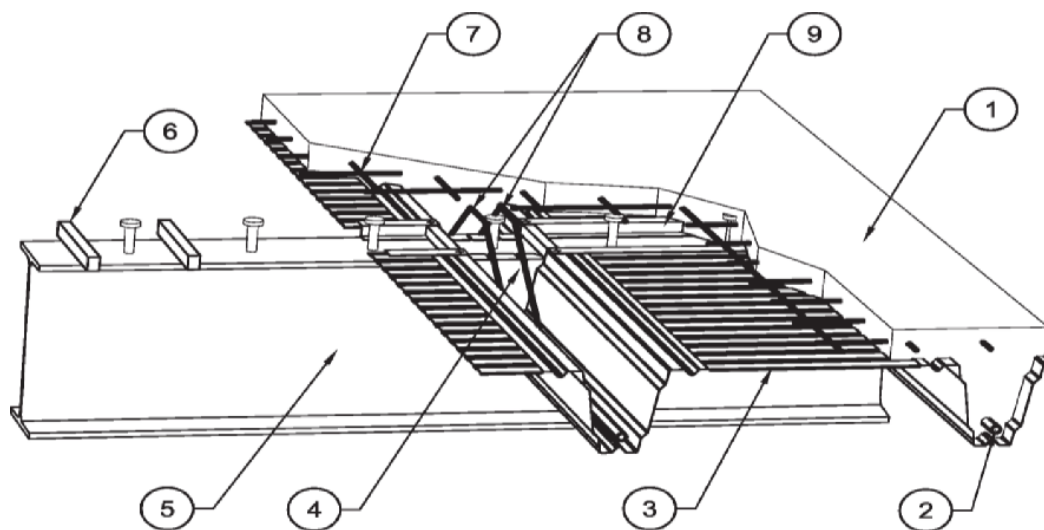
appropriate erection load in accordance with section 3.3.1, are avoided.

The formation of the floor shaped sheet metal parts for production of continuity affect, when concreting in the areas of the shaped sheet metal part, attention must be paid to a careful compaction of the concrete and complete concrete filling.

The compliance of the execution of the floor systems produced with the profile panels with the provisions of this National Technical Approval must be certified in writing by each of the construction companies. This declaration must be submitted to the building owner for possible required forwarding to the responsible building supervisory authority.

Andreas Schult
Head of unit

Legally attested

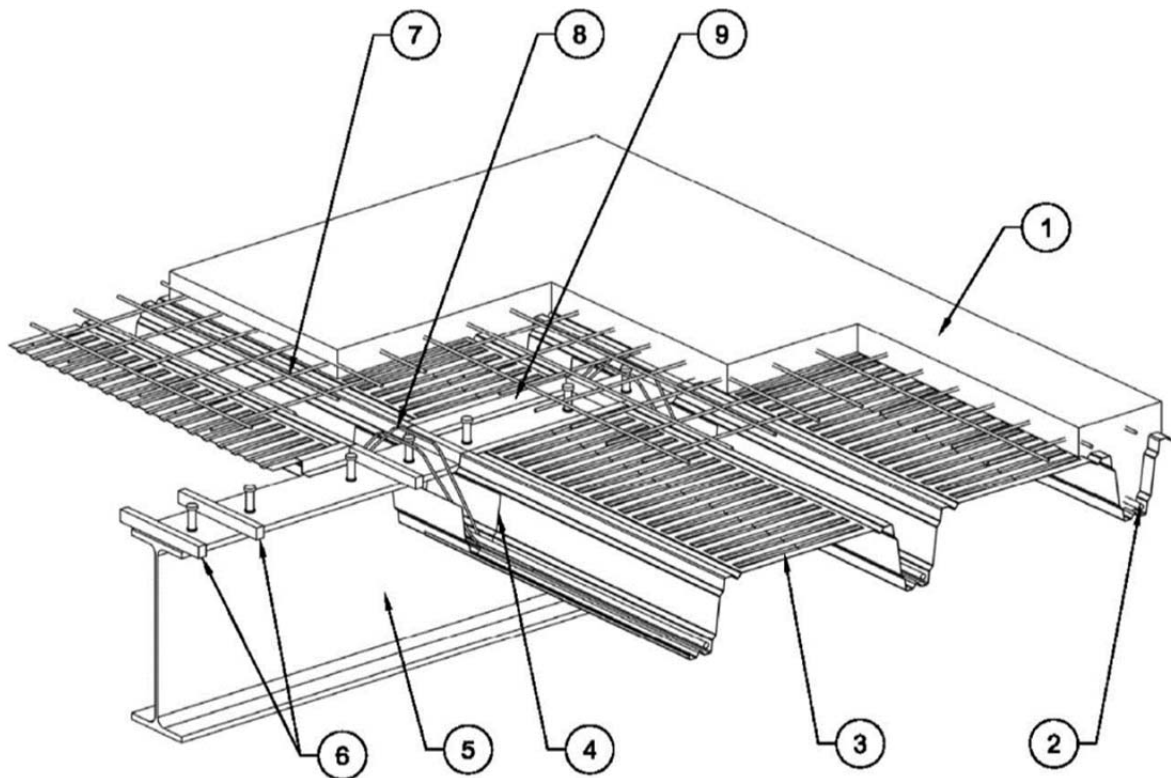


- 1 Concrete (at least C20/25)
- 2 Rip reinforcement
- 3 Trapezoidal profile panel (steel)
- 4 Cover cap (plastic)
- 5 Composite steel girder
- 6 Support cleats (steel)
- 7 Floor reinforcement
- 8 Design support reinforcement
- 9 Z-profile (sheet steel profile)

Hoesch Additive Floor®

System overview
of the variant "chain of single span girders"

Annex 1

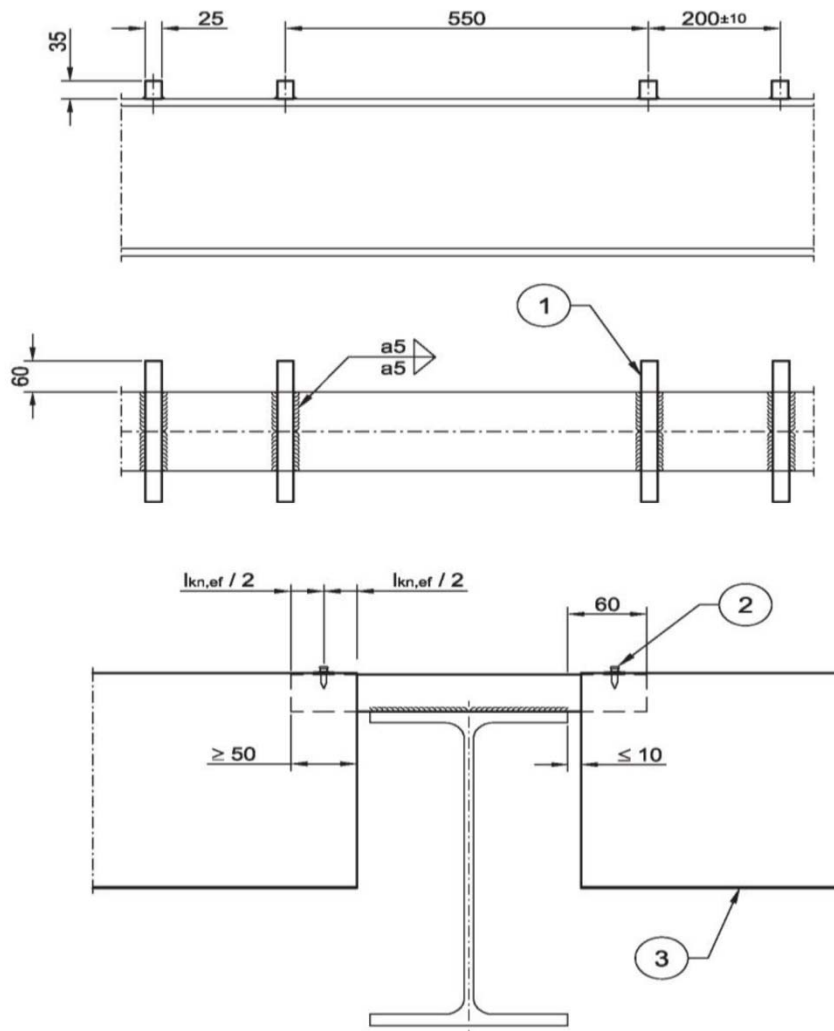


- 1 Concrete (at least C20/25)
- 2 Rip reinforcement
- 3 Trapezoidal profile panel (steel)
- 4 Shaped sheet metal part (dimensions on file at DIBt)
- 5 Composite steel girder
- 6 Support cleats (steel)
- 7 Floor reinforcement
- 8 Design support reinforcement
- 9 Z-profile (sheet steel profile)

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System overview
of the variant "continuous girder"

Annex 2



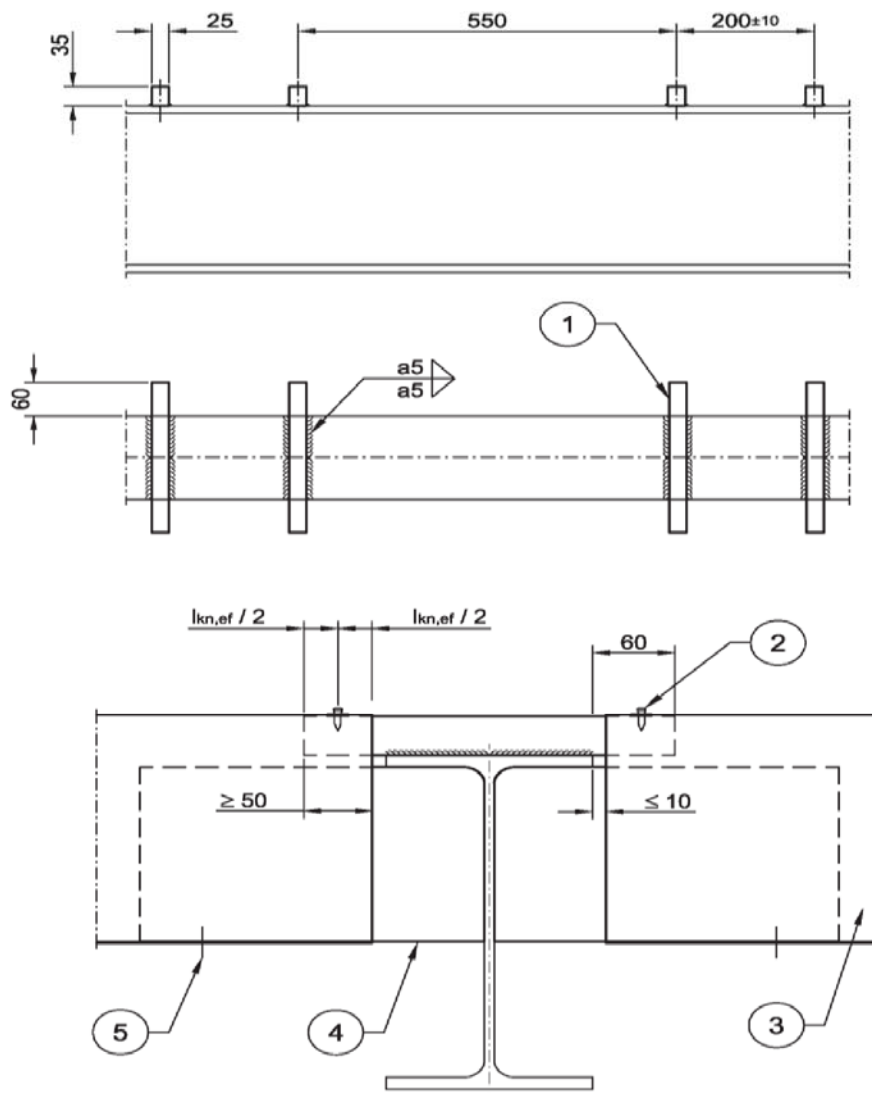
- 1 Steel cleat S 235
- 2 Cartridge-fired pins (with National Technical Approval)
- 3 Trapezoidal profile panel TRP 200

All dimensions in [mm]

Hoesch Additive Floor®

Support cleats on the intermediate support
of the variant "chain of single span girders"

Annex 3



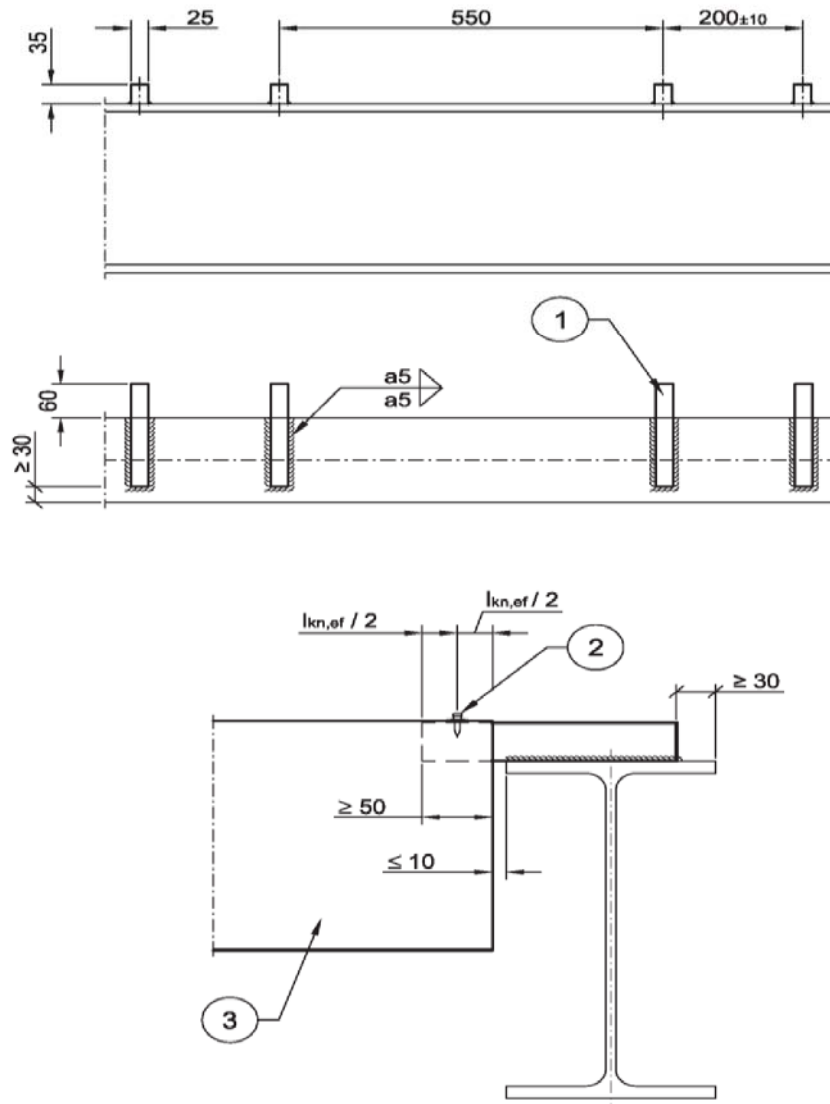
- 1 Steel cleat S 235
- 2 Cartridge-fired pins (with National Technical Approval)
- 3 Trapezoidal profile panel TRP 200
- 4 Shaped sheet metal part
- 5 Fasteners with National Technical Approval

All dimensions in [mm]

Hoesch Additive Floor®

Support cleats on the intermediate support
of the variant "continuous girder"

Annex 4



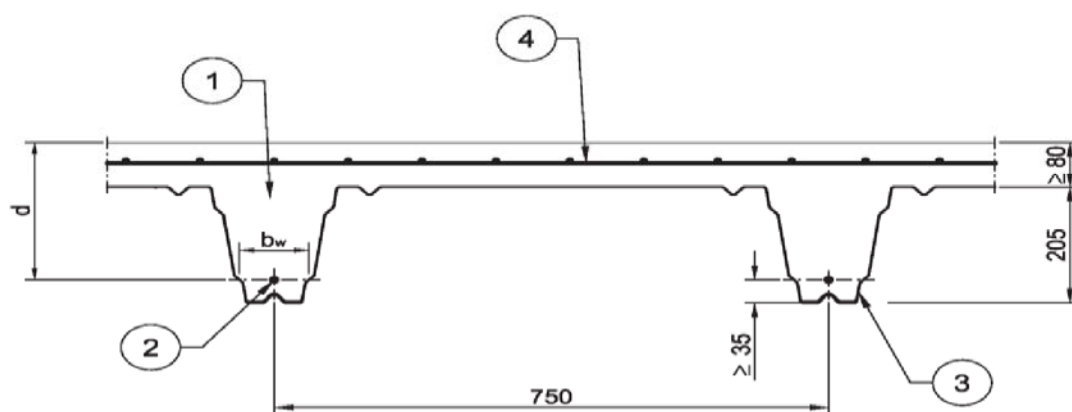
- 1 Steel cleat S 235
- 2 Cartridge-fired pins (with National Technical Approval)
- 3 Trapezoidal profile panel TRP 200

All dimensions in [mm]

Hoesch Additive Floor®

Support cleats on the end support

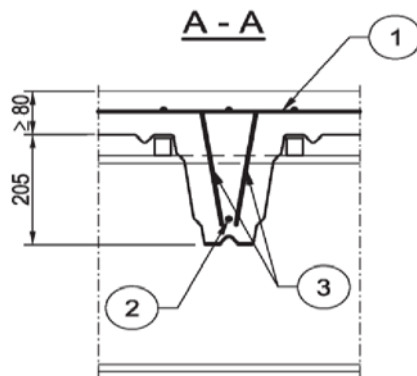
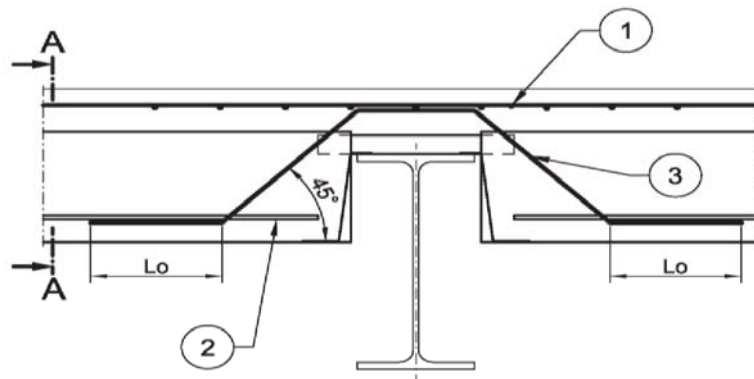
Annex 5



- 1 Concrete (at least C 20/25)
- 2 Rip reinforcement
- 3 Trapezoidal profile panel TRP 200
- 4 Floor reinforcement

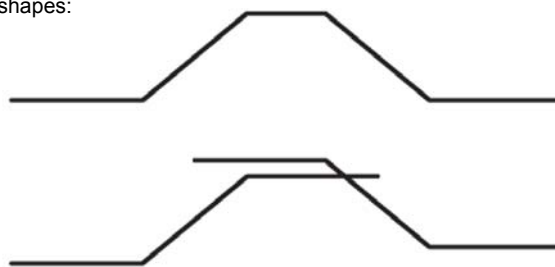
All dimensions in [mm]

Hoesch Additive Floor®	Annex 6
Floor cross-section	



- 1 $A_{s, \text{ floor}}$
- 2 $A_{a, \text{ rib}}$
- 3 $A_{s, \text{ stirrup}}$ (at least 2 stirrups $\varnothing 6$ BST 500 S)
- L_o Lap length in accordance with DIN EN 1992-1-1

Possible stirrup shapes:

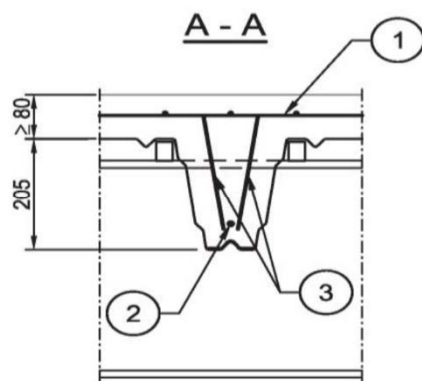
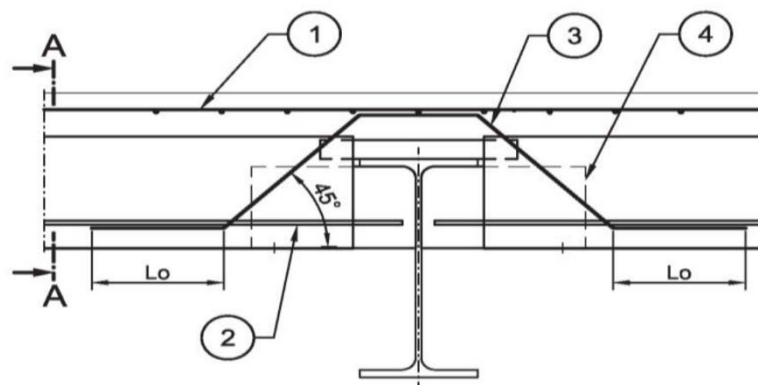


All dimensions in [mm]

Hoesch Additive Floor®

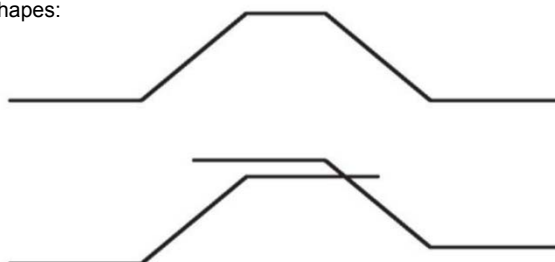
Static required reinforcement on the intermediate support
of the variant "chain of single span girders"

Annex 7



- 1 $A_{s, \text{ floor}}$
- 2 $A_{s, \text{ rib}}$
- 3 $A_{s, \text{ stirrup}}$ (at least 2 stirrups $\varnothing 6$ BST 500 S)
- 4 Shaped sheet metal part
- L_o Lap length in accordance with DIN EN 1992-1-1

Possible stirrup shapes:

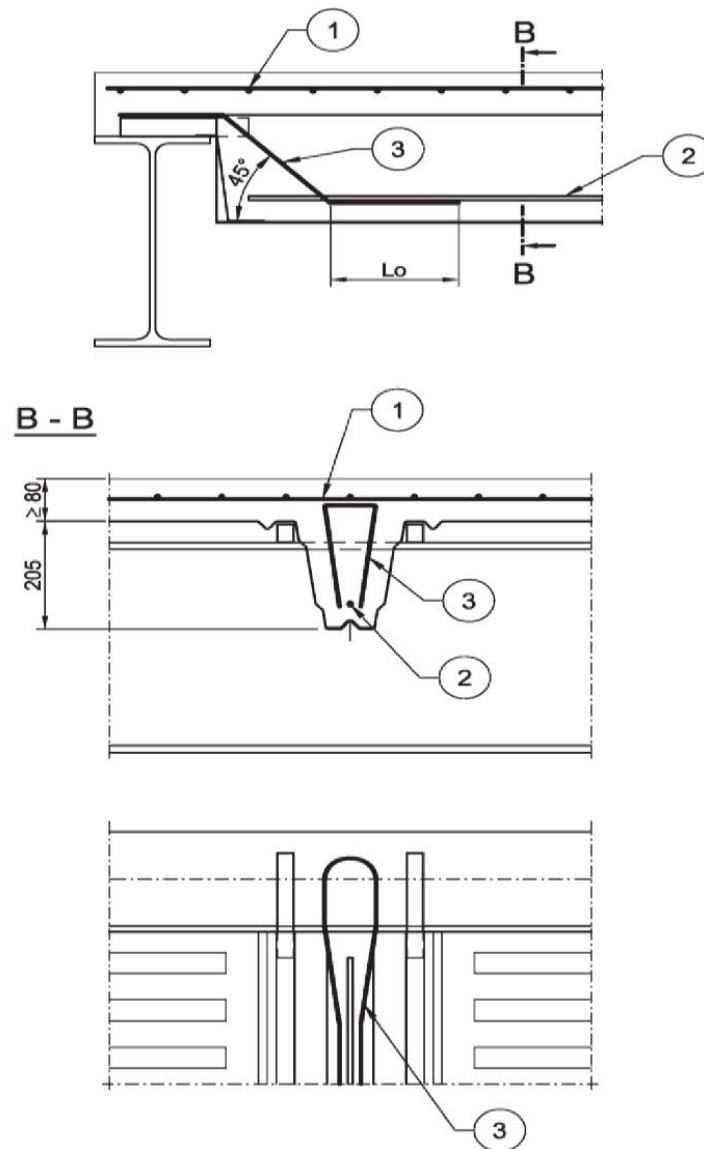


All dimensions in [mm]

Hoesch Additive Floor®

Static required reinforcement on the end support
of the variant "chain of single span girders"

Annex 8



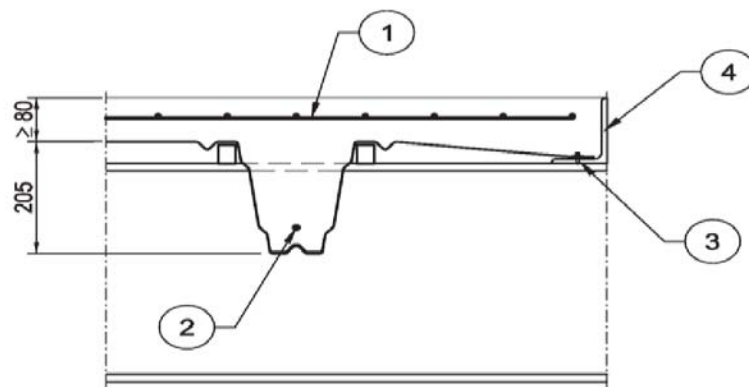
- 1 $A_{s, \text{floor}}$
2 $A_{s, \text{rib}}$
3 $A_{s, \text{stirrup}}$ (1 Bü Ø6 BST 500 S)
 L_o Lap length in accordance with DIN EN 1992-1-1

All dimensions in [mm]

Hoesch Additive Floor®

Static required reinforcement on the end support
of the variant "chain of single span girders"

Annex 9



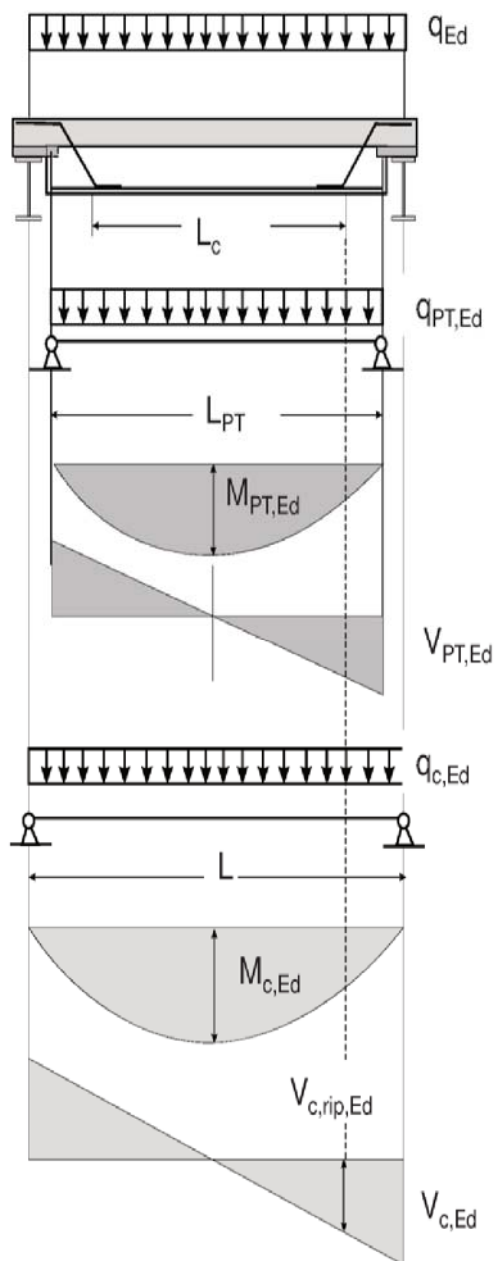
- 1 $A_{s, \text{floor}}$
- 2 $A_{s, \text{rib}}$
- 3 Cartridge-fired pins (with National Technical Approval)
- 4 Contact angle

All dimensions in [mm]

Hoesch Additive Floor®

Edging

Annex 10



Hoesch Additive Floor®

Calculation model for verification of transverse tensile capacity

Annex 11

Table 1: Cross-section values and characteristic resistance variables of the profiled metal sheets

Cross-section values			Characteristic values of the moment load capacity
Nominal sheet metal thickness	Concrete quality	Moment of inertia	Downward directed, uniformly distributed load
t_{nom}	g	I_{ef}	$M_{\text{pT.Rk}}$
[mm]	[kN/m ²]	[cm ⁴ /m]	[kNm/m]
1.00	0.128	653	17.0
1.13	0.145	758	19.9
1.25	0.160	855	22.1
1.50	0.192	1030	26.5

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Cross-section values and characteristic resistance variables of the profiled metal sheets

Annex 12

Table 1: Factors k for consideration of the load-bearing proportion of the individual components

Component	Factor	Edge supports	Intermediate support in accordance with Annex 3	Intermediate support in accordance with Annex 4
Top concrete layer	k_c	0.50	0.60	0.30
Suspended reinforcement	k_s	0.85	1.00	1.00

Table 2: Calculated value $V_{Rd,c,min}$ of the transverse tensile capacity component of the reinforced concrete rib in [kN]

Transverse load capacity [kN]	Top concrete layer thickness h_c [cm]	Concrete strength class					
		C 20/25	C 25/30	C 30/37	C 35/45	C 40/50	C 45/55
$V_{Rd,c}$	8 cm	10.1	11.3	12.3	13.3	14.2	15.1
	9 cm	10.3	11.6	12.7	13.7	14.6	15.5
	10 cm	10.6	11.8	13.0	14.0	15.0	15.9

Table 3: Calculated value $V_{Rd,s}$ of the transverse tensile capacity component of the shear reinforcement of a rib in [kN] on the intermediate support

Transverse load capacity [kN]	Reinforcement diameter ϕ [mm]				
	6	7	8	9	10
$V_{Rd,s}$	17.4	23.6	31.0	39.1	48.3

Table 4: Calculated value $V_{Rd,s}$ of the transverse tensile capacity component of the shear reinforcement in [kN] on the intermediate support

Transverse load capacity [kN]	Concrete strength class	Reinforcement diameter ϕ [mm]				
		6	7	8	9	10
$V_{Rd,s}$	C 20/25	8.6	10.1	11.5	13.0	14.4
	C 25/30	10.0	11.7	13.4	15.0	16.7
	C 30/37	11.3	13.2	15.1	17.0	18.9
	C 35/45	12.5	14.6	16.7	18.8	20.9
	C 40/50	13.0	16.0	18.3	20.6	22.8
	C 45/55	13.0	17.3	19.8	22.2	24.7

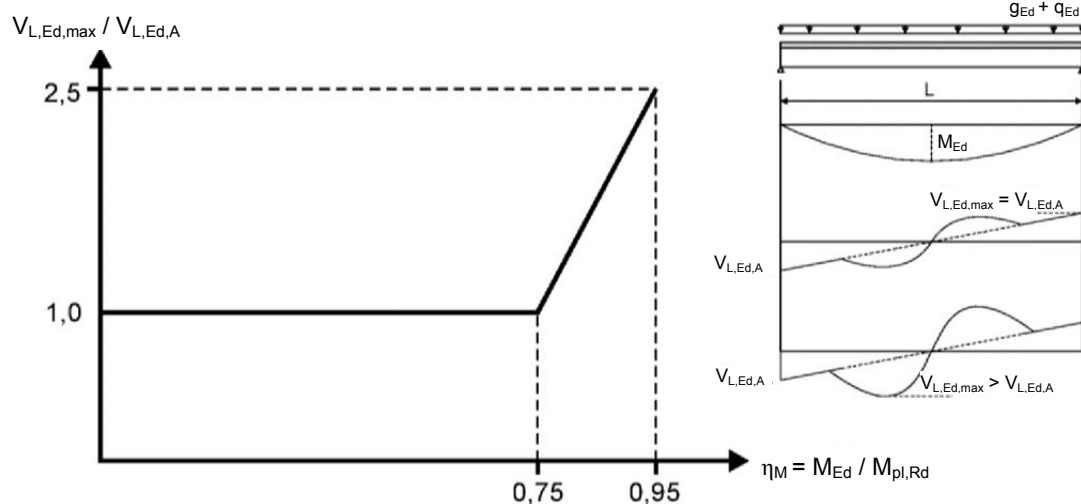
Table 5: Calculated value $V_{Rd,K}$ of the transverse tensile capacity component of a rib of the trapezoidal sheet in [kN]

Transverse load capacity [kN]	Nominal thickness of the trapezoidal sheet t_{nom} [mm]			
	1.00	1.13	1.25	1.50
$V_{Rd,K}$	13.5	16.2	19.0	25.2

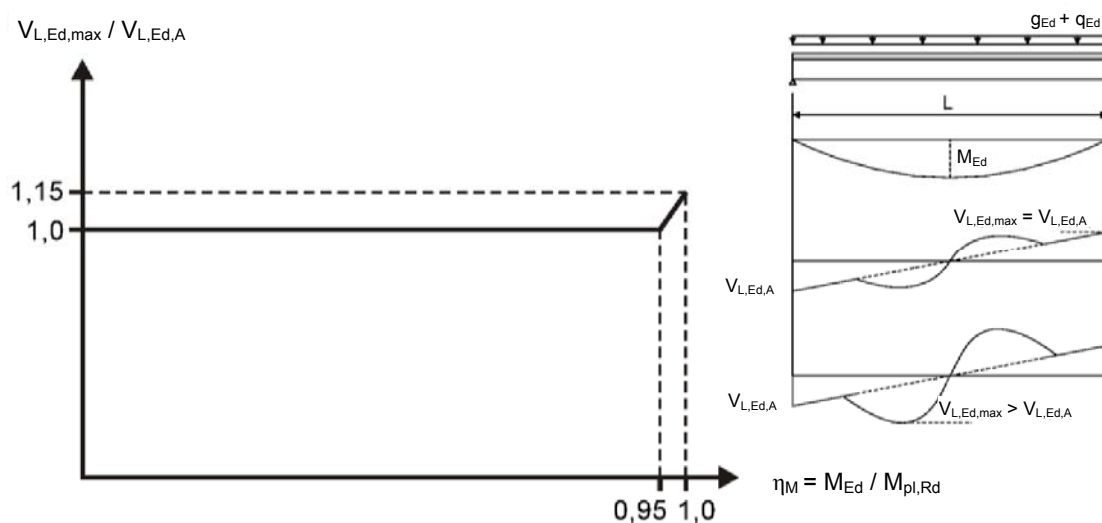
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Calculated value of the transverse tensile capacity in the support area

Annex 13



Calculation diagram for determination of the maximum longitudinal shear force at nonlinear dimensioning of the composite steel girder of the Hoesch Additive Floor® without inherent weight composite

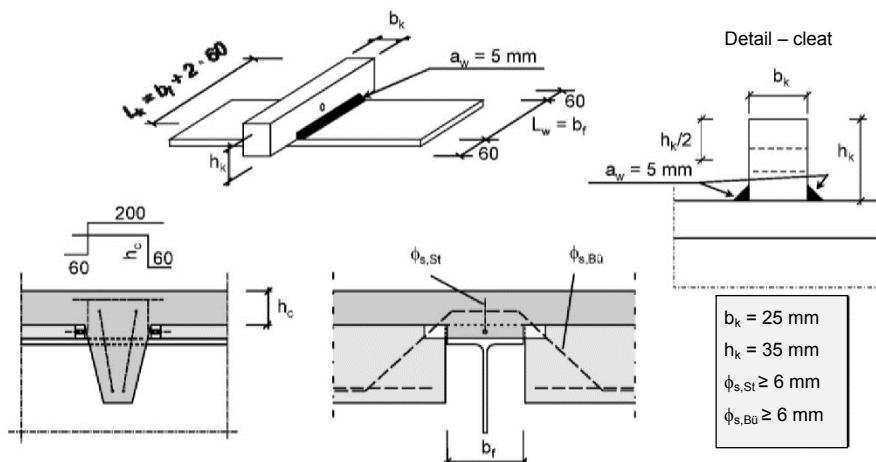


Calculation diagram for determination of the maximum longitudinal shear force at nonlinear dimensioning of the composite steel girder of the Hoesch Additive Floor® with inherent weight composite

Hoesch Additive Floor®

Diagrams for the non-linear calculation of the composite steel girder

Annex 14



Boundary design conditions for the execution of the cleats

Longitudinal thrust load capacity: $V_{L,Rd} = \frac{n}{e} \cdot P_{Rd}$

with $P_{Rd} = P_{Rd,c} + P_{Rd,s}$ longitudinal shear force capacity per cleat
 $n = 2$ number of cleats per reinforced concrete rib
 $e = 0.75 \text{ m}$ spacing of reinforced concrete ribs

Table 1: Component of the calculated value of the longitudinal shear force load capacity arising from the shear strength of the concrete $P_{Rd,c}$ [kN] based on a cleat

Flange width b_f [mm]	Concrete quality					
	C 20/25	C 25/30	C 30/37	C 35/45	C 40/50	C 45/55
200	103.2	119.7	135.2	149.8	163.7	177.1
250	112.8	130.9	147.8	163.8	179.1	193.7
300	122.5	142.1	160.5	177.9	194.4	210.3

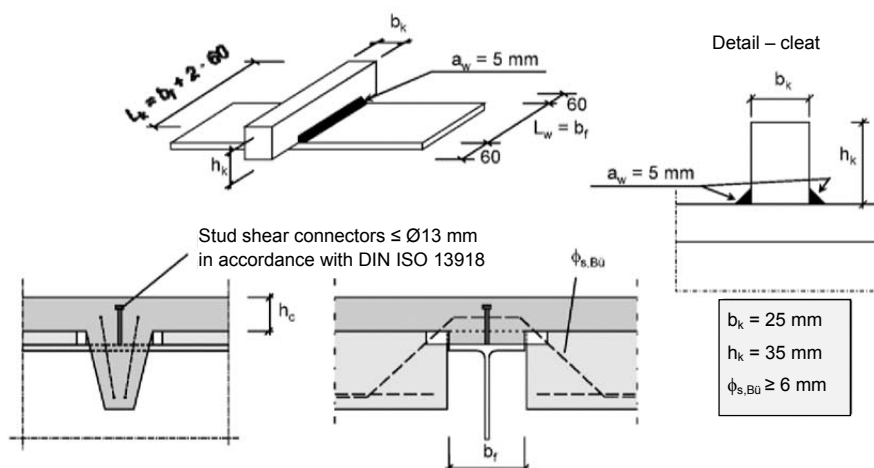
Table 2: Component of the calculated value of the longitudinal shear force load capacity arising from the shear force intersecting reinforcement $P_{Rd,s}$ [kN] based on a cleat.

Diameter $\phi_{s,st}$ of the connectors in the cleats [mm]	6	8	10
$P_{Rd,s}$ [kN]	10.4	18.6	29.0

Hoesch Additive Floor®

Longitudinal shear force capacity in the composite joint of the composite steel girders when using stirrups

Annex 15



Boundary design conditions for the execution of the cleats

Longitudinal thrust load capacity: $V_{L,Rd} = \frac{n}{e} \cdot P_{Rd}$

with $P_{Rd} = P_{Rd,c} + P_{Rd,z}$ longitudinal shear force capacity per cleat
 $n = 2$ number of cleats per reinforced concrete rib
 $e = 0.75 \text{ m}$ spacing of reinforced concrete ribs

Table 1: Component of the calculated value of the longitudinal shear force load capacity arising from the shear strength of the concrete $P_{Rd,c}$ [kN] based on a cleat

Flange width b_f [mm]	Concrete quality					
	C 20/25	C 25/30	C 30/37	C 35/45	C 40/50	C 45/55
200	103.2	119.7	135.2	149.8	163.7	177.1
250	112.8	130.9	147.8	163.8	179.1	193.7
300	122.5	142.1	160.5	177.9	194.4	210.3

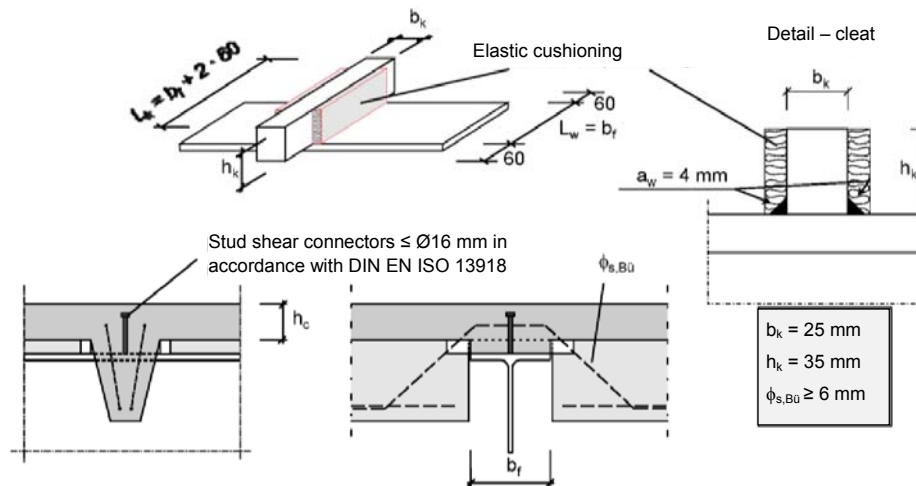
Table 2: Component of the calculated value of the longitudinal shear force load capacity arising from the shear force intersecting tension anchors $P_{Rd,z}$ [kN] based on a cleat

Diameter d_z tension anchors [mm]	10	13
$P_{Rd,z}$ [kN]	7.0	11.8

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Longitudinal shear force capacity in the composite joint of the composite steel girders when using tension anchors

Annex 16

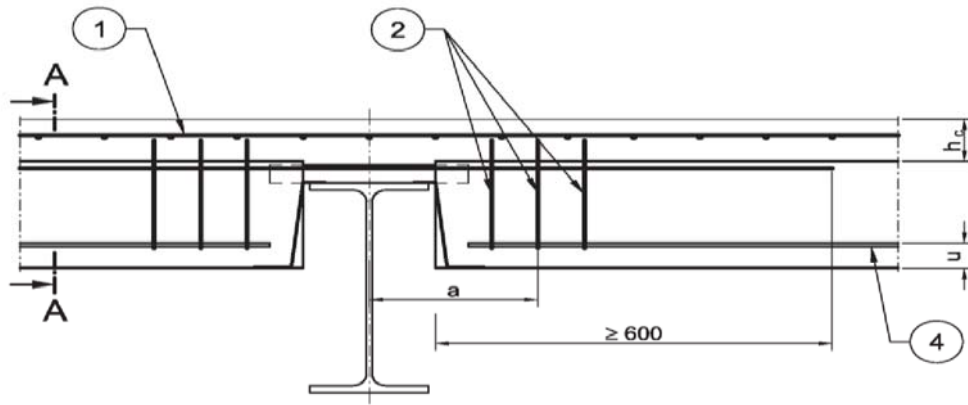


Required cushioning of the cleats for systematic use of ductile composite materials

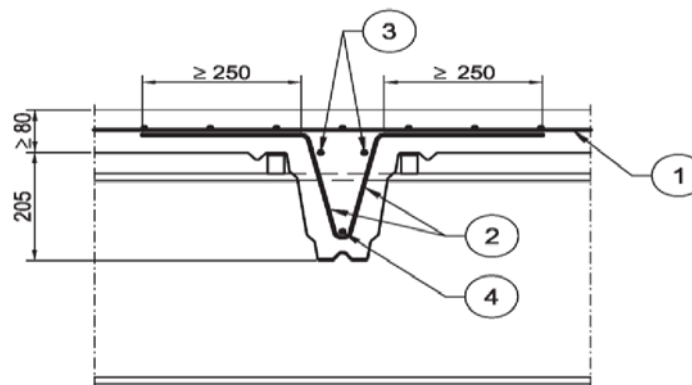
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Required cushioning of the cleats for systematic use of ductile stud shear connectors

Annex 17



A - A



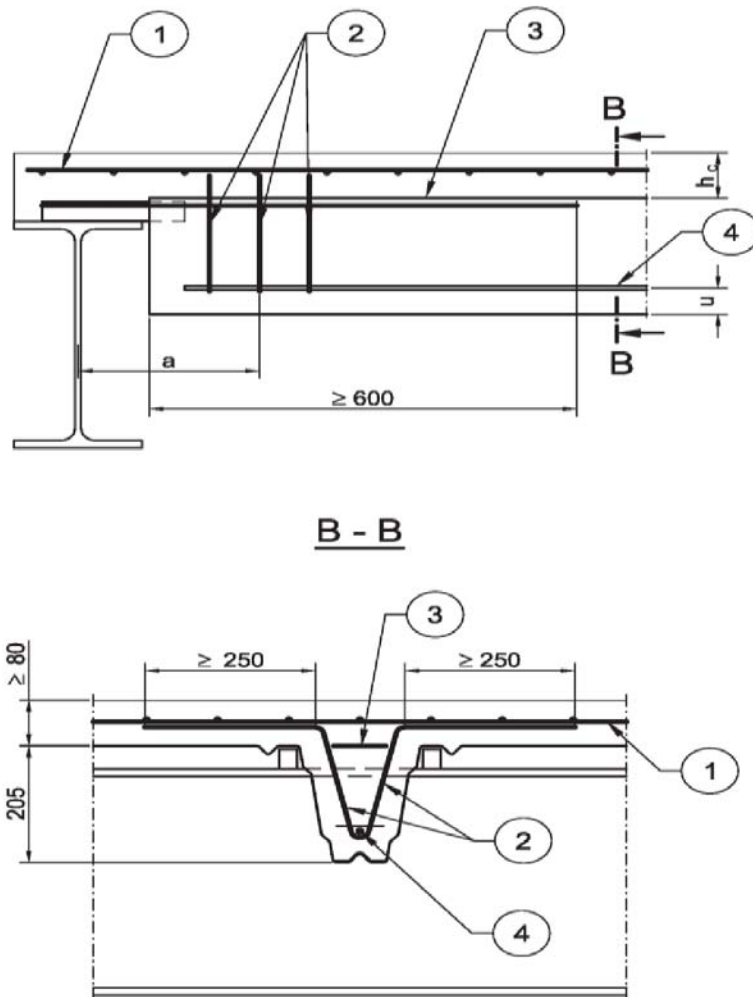
- 1 $A_{s, \text{floor}}$
- 2 Vertical suspended reinforcement $A_{s, V}$
in the form of a curved stirrup mesh
- 3 Horizontal suspended reinforcement $A_{s, H}$
- 4 $A_{s, \text{rib}}$

All dimensions in [mm]

Hoesch Additive Floor®

Fire protection suspended reinforcement on the intermediate support

Annex 18



- 1 $A_{s, \text{floor}}$
- 2 Vertical suspended reinforcement $A_{s, V}$
in the form of a curved stirrup mesh
- 3 Horizontal suspended
reinforcement $A_{s, H}$ as loop
- 4 $A_{s, \text{rib}}$

All dimensions in [mm]

Hoesch Additive Floor®

Fire protection suspended reinforcement on the end support

Annex 19