# The Second Generation Eurocodes

Key changes and benefits through design examples

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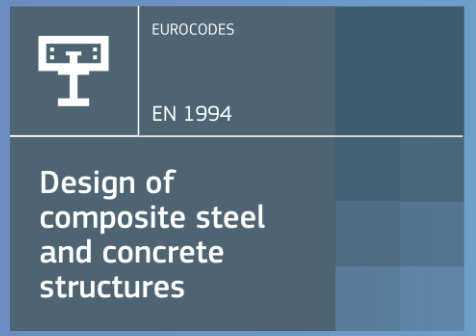
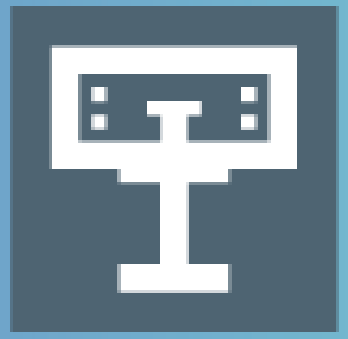
# EN 1994 ‘Eurocode 4: Design of composite steel and concrete structures’

Stephen Hicks, Mike Banfi, Wolfgang Kurz, François Hanus; CEN/TC 250/SC 4



# Eurocode 4 Design of composite steel and concrete structures

## EN 1994 (1st generation or 1G)

- EN 1994-1-1, Part 1-1: General rules and rules for buildings.
- EN 1994-1-2, Part 1-2: General rules – Structural fire design.
- EN 1994-2, Part 2: General rules and rules for bridges

## EN 1994 (2nd generation or 2G)

- EN 1994-1-1, Part 1-1: General rules and rules for buildings.
- EN 1994-1-2, Part 1-2: Structural fire design.
- EN 1994-2, Part 2: Bridges.
- CEN/TS 1994-1-101 Double and single skin steel concrete composite (SC) structures.
- CEN/TS 1994-1-102 Design rules for the use of Composite Dowels.
- CEN/TS 1994-1-103 Design rules for high performance composite columns.

# EN 1994-1-1

*Existing Annexes → Normative*

*Existing Annexes → Informative*

*New Annexes → Normative*

*New Annexes → Informative*

1. Scope
2. Normative references
3. Terms, definition and symbols
4. Basis of design
5. Materials
6. Durability
7. Structural analysis
8. Ultimate limit states
9. Serviceability limit states
10. Composite slabs with profiled steel sheeting for buildings
11. Composite joints in frames for buildings

A. Stiffness of joint components in buildings

B. Standard tests

C. Shrinkage of concrete for composite structures for buildings

D. Composite beams with web openings

E. Composite beams with web openings and a stiff slab

F. Headed studs that cause splitting forces in the direction of the slab thickness

G. Design resistance of headed studs used with open trough profiled steel sheeting in buildings with ribs transverse to the supporting beams

H. Design tension resistance of headed studs

I. Additional rules for shallow floor beams

J. Other flooring types using precast concrete slabs in buildings

# EN 1994-2

1. Scope
2. Normative references
3. Terms, definition and symbols
4. Basis of design
5. Materials
6. Durability
7. Structural analysis
8. Ultimate limit states
9. Serviceability limit states
10. Precast concrete slabs
11. Composite plates

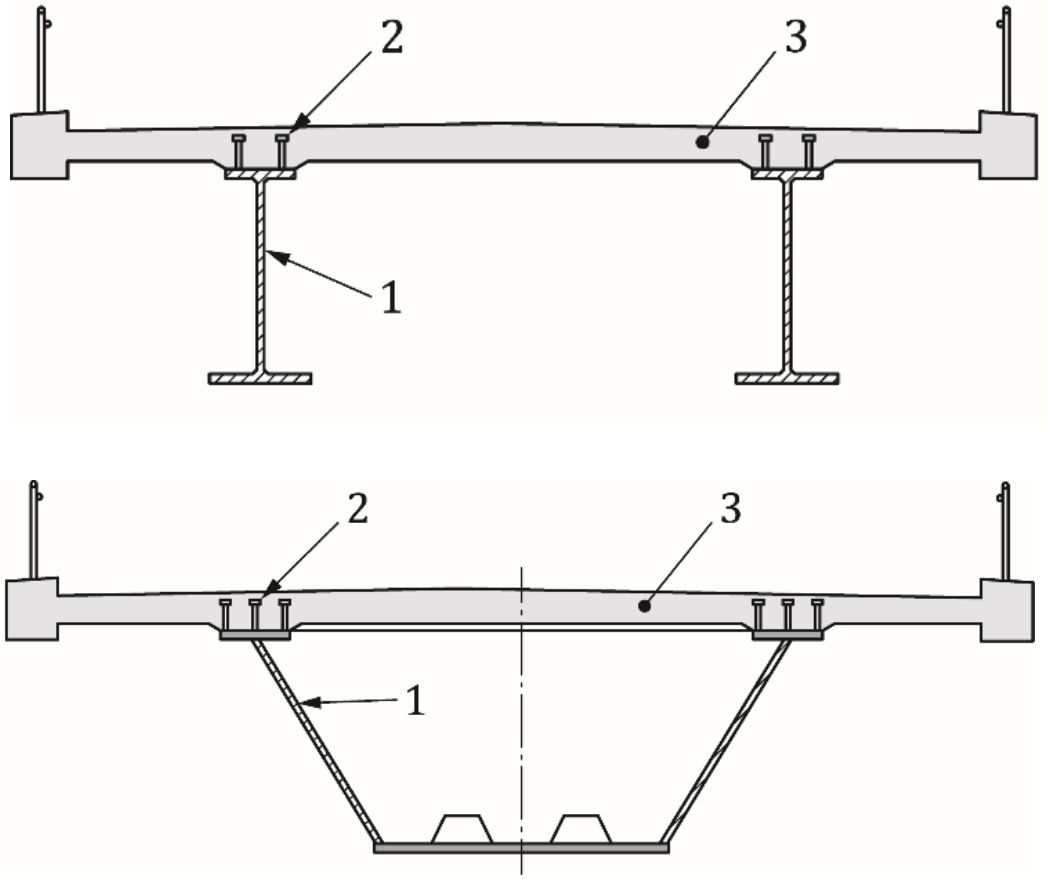
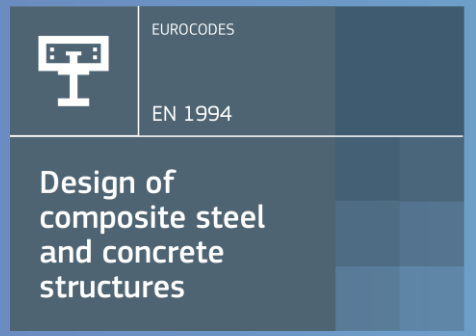
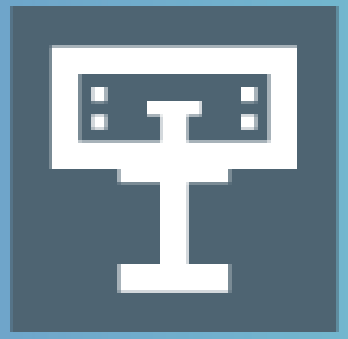



*Figure 8.1 © BSI

# Part 1-1

Stephen Hicks, Mike Banfi, Wolfgang Kurz, François Hanus; CEN/TC 250/SC 4

# Materials: Steel, concrete and reinforcing steel

- EN 1993-1-1: 2023
  - Structural steel grades between S235 and S700
- EN 1992-1-1: 2023
  - Concrete strength classes between:
    - C12/15 and C100/115 normal weight concrete
    - LC12/13 and LC80/88 lightweight concrete
  - Reinforcing steel strength classes between B400 and B700
- EN 1994-1-1
  - Design method based on plastic global analysis and plastic resistance:
    - Reinforcing steel strength classes not greater than B500 for continuous composite beams, columns, and slabs.
    - Structural steel with a nominal yield strength not more than $f_y$ = 460 MPa.
  - Concrete strength classes limited to C20/25 and C70/85 (previously C60/75), together with LC20/22 and LC60/66.
  - When design method based on elastic or strain-limited design principles, full range of reinforcing and structural steel grades in EN 1992-1-1 & EN 1993-1-1, may be used.

# Materials: Design compressive strength of concrete

According to EN 1992-1-1, design compressive strength of the concrete $f_{cd}$ should be taken as:

$$f_{cd} = \eta_{cc} k_{tc} \frac{f_{ck}}{\gamma_C}$$

where
$\eta_{cc}$ is a factor to be taken from EN 1992-1-1, unless ‡ below applies
$k_{tc}$ is a factor to be taken from EN 1992-1-1 considering the effect of high sustained loads and of time of loading on concrete compressive strength, unless ‡ below applies
$f_{ck}$ is the characteristic compressive strength of the concrete in accordance with EN 1992-1-1
$\gamma_C$ is the partial factor for concrete.

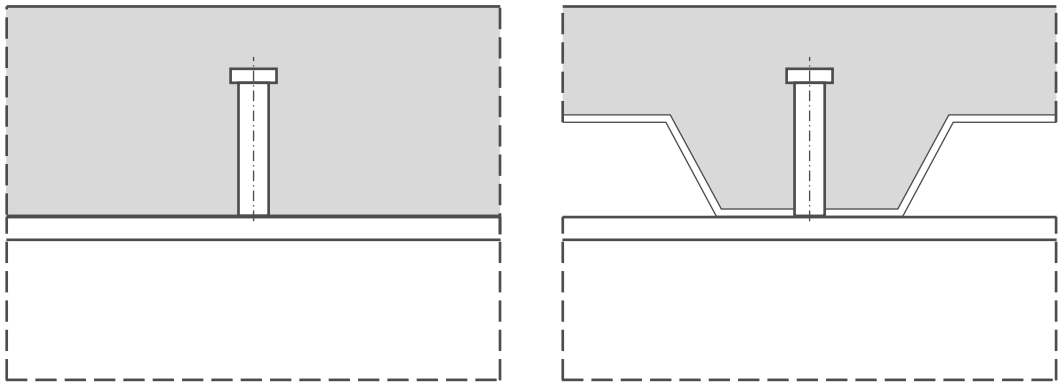
‡ For composite structures, $k_{tc}\eta_{cc}$ should not exceed 0,85 when:

- plastic resistance based on rectangular stress block; or
- bending resistance determined using non-linear methods, where the complete thickness of the concrete flange is subject to compression.

For the plastic resistance of concrete-filled steel tubes, $k_{tc}\eta_{cc}$ may be taken as 1,0 for a concrete strength class up to C50/60 and $t_{ref} \geq 28$ days

# Materials: Ductility Categories for shear connectors

| Ductility Category | Characteristic elastic slip when $P_{Rk}$ is reached $\delta_{ek}$ (mm) | Characteristic slip capacity $\delta_{uk}$ (mm) |
|---|---|---|
| D1 | - | - |
| D2 | $\delta_{ek} \leq 2{,}5$ | $6{,}0 \leq \delta_{uk} < 10{,}0$ |
| D3 | | $\delta_{uk} \geq 10{,}0$ |



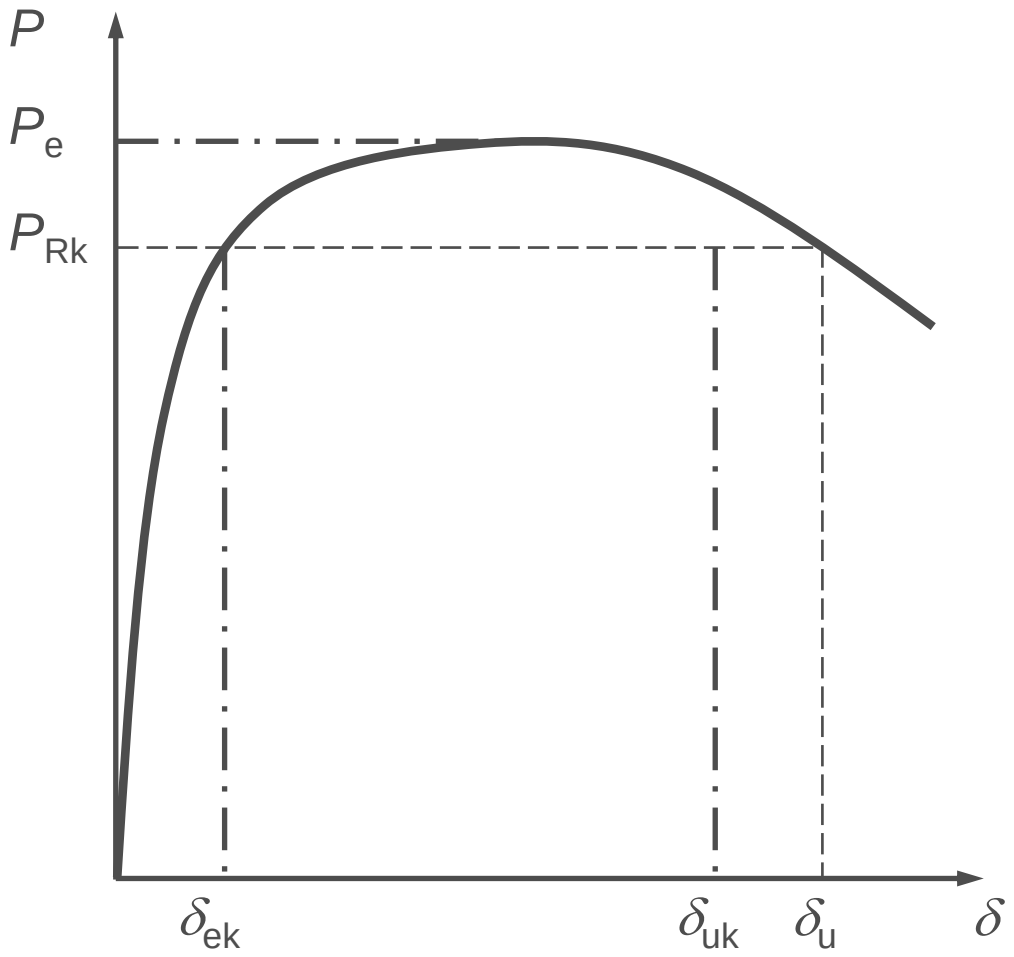



*Figure B.3 © BSI

# Section 8: Ultimate limit states: Design resistance of headed studs in solid slabs and concrete encasements

When the height of the stud $h_{sc} \geq 3.9d$, design resistance determined from minimum of:

$$P_{Rd,s} = \frac{0{,}8 f_u \pi d^2/4}{\gamma_V} \text{ or } P_{Rd,c} = \frac{0{,}29 k_{cc} d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_V}$$

where
$d$ is the diameter of the shank of the stud, 16 mm ≤ $d$ ≤ 25 mm;
$f_u$ is the specified ultimate tensile strength of the material of the stud but not greater than 500 N/mm$^2$;
$f_{ck}$ is the characteristic cylinder compressive strength of the concrete at aged considered, density ≥ 1750kg/m$^3$;
$E_{cm}$ is the secant modulus of elasticity of the concrete for short-term loading according to EN 1992-1-1, where coefficient $k_E \leq 9500$
$\gamma_V$ is the partial factor, which is an NDP ($\gamma_V$ is 1,25 unless NA gives a different value)
$k_{cc}$ is a reduction factor considering the effect of concrete relaxation under sustained loading, which is an NDP ($k_{cc}$ is 1,0 unless NA gives a different value).

Reduction factor $\alpha$ previously applied to $P_{Rd,c}$ has been removed, as it was based on a rather small number of push tests from the 1960's, and its exclusion improves the reliability of the design model.

# Section 8 and Annex F: Headed studs that cause splitting forces in the direction of the slab thickness

When splitting forces can occur in direction of slab and there is no transverse shear, design resistance determined as min{$P_{Rd,s}$; $P_{Rd,C}$} when:

- fatigue is not relevant;
- transverse reinforcement provided with $a_{rp}$ ≥6$d$, and v ≥4$d$; and
- splitting force is resisted by stirrups designed for tensile force of 0,3$P_{Rd}$ per stud, with spacing of stirrups ≤ 18$d$ and longitudinal spacing of studs

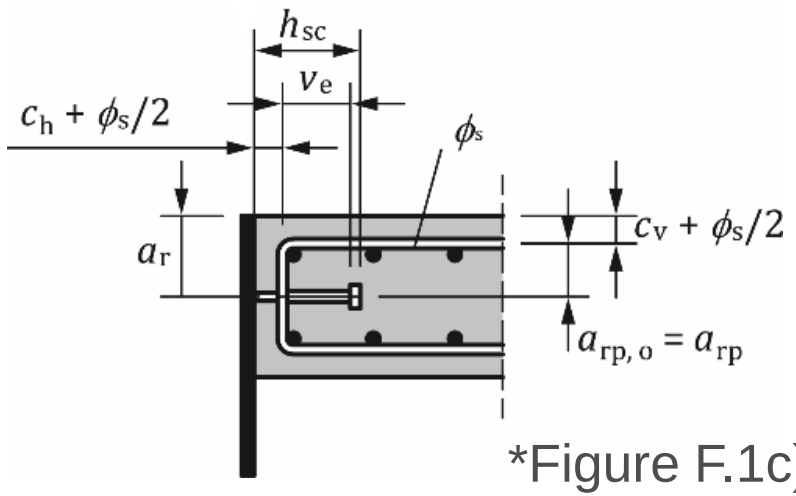



*Figure F.1c) © BSI

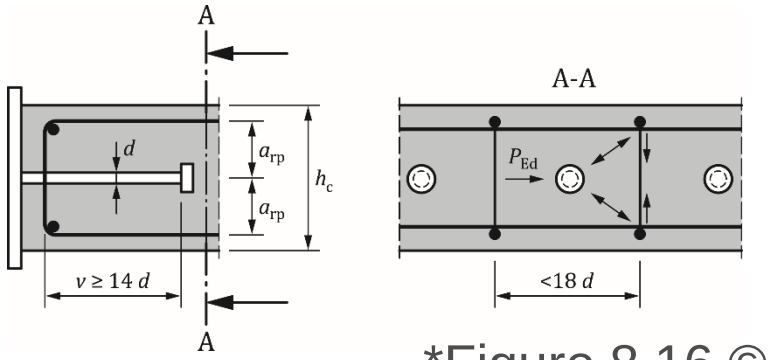



*Figure 8.16 © BSI

When studs close to the concrete edge design resistance min{$P_{Rd,s}$; $P_{Rd,C}$; $P_{L,Rd}$}

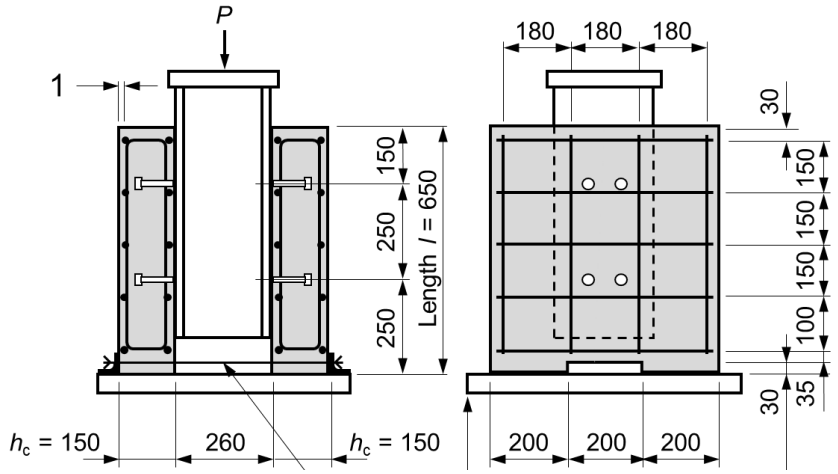
To prevent pull-out, required anchorage length $v$ should satisfy following conditions

- uncracked concrete $v$ ≥ max{110 mm; 1,7$a_{rp}$; 1,7 $s$/2}
- cracked concrete $v$ ≥ max{160 mm; 2,4$a_{rp}$; 2,4 $s$/2}

When shorter studs provided ($v_e$ ≤ $v$), new reduction factor $\beta_s$ given which should be applied to $P_{L,Rd}$

# Annex B: Standard push test

*Figure B.1 © BSI




- Recess no longer optional
- For floor beams, optional tension ties now included.
- Long-term effects considered by applying load to specimen in 1.0 mm increments of slip and paused for at least 5 minutes to enable load relaxation to be determined

- For floor beams, a normal force may be applied as an alternative to the optional tie
- Long-term effects, identical loading protocol as the standard test.
- Recommended value for $K_p$ is 0.1 (NDP)

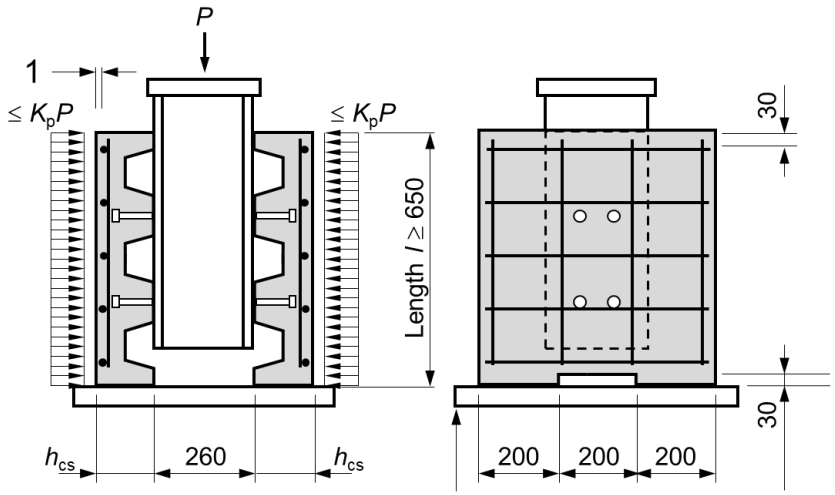



*Figure B.2 © BSI

# Annex G: New rules for headed studs in sheeting with ribs transverse to beam

Source : Vigneri V *et al.* (2022), Design models for predicting the resistance of headed studs in profiled sheeting, Steel and Composite Structures, 42(5), 633-647

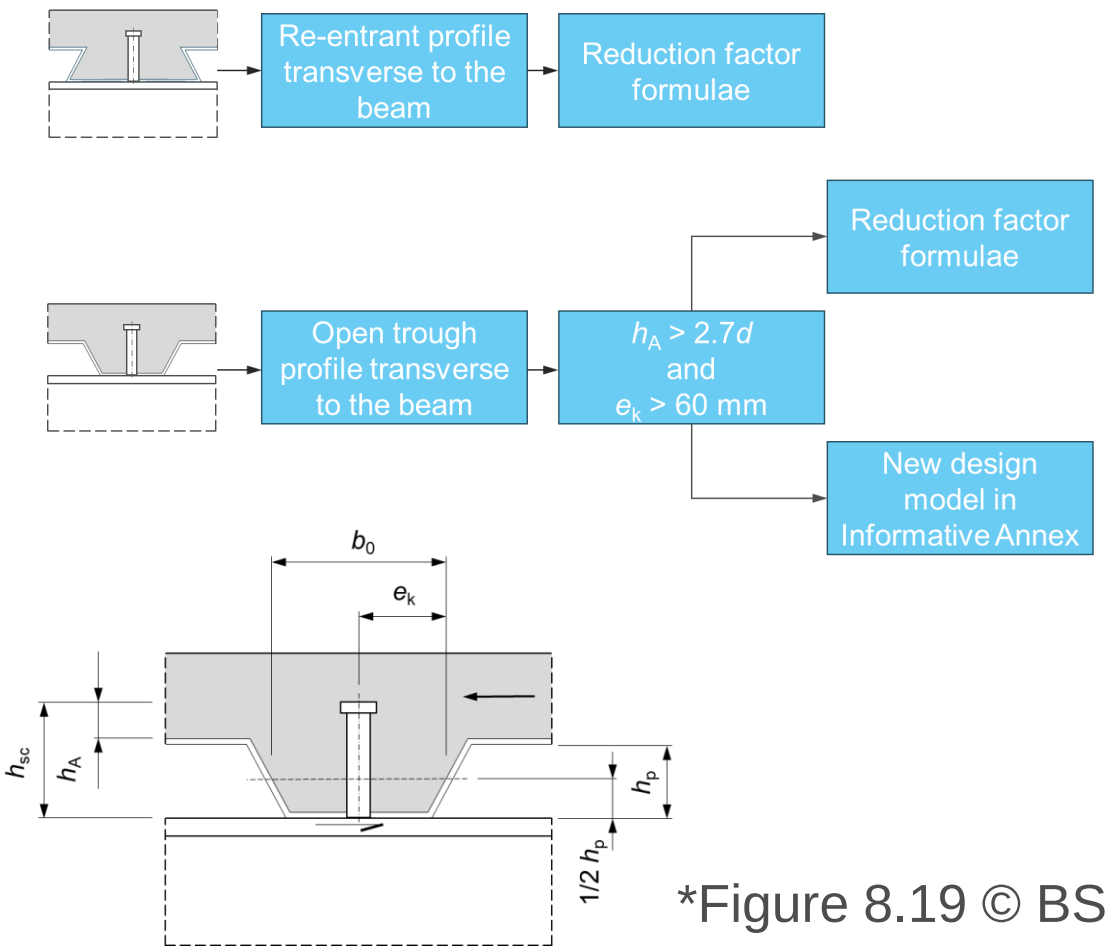
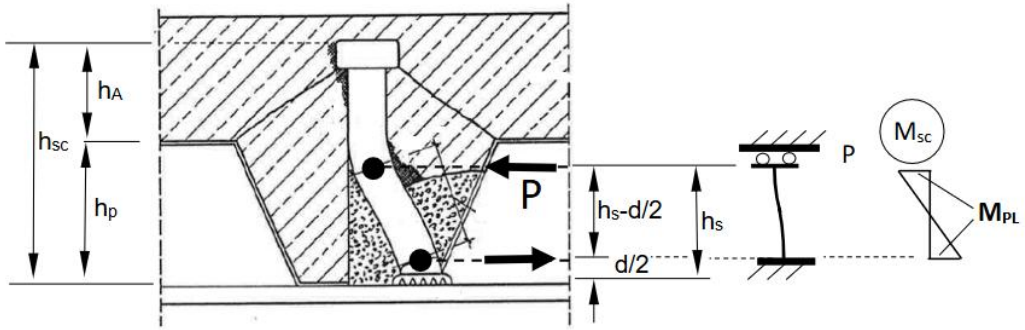



*Figure 8.19 © BSI

- For ribs transverse to the supporting beam, current 1G EN 1994 design model is to multiply resistance of a stud in a solid slab by a reduction factor $k_t$
- Research has shown that the reduction factor equations do not perform well for modern open trough profiles
- New design model in Informative Annex based on a mechanical cantilever model
- Geometries where reduction factor applies deemed to be at least Ductility Category D2 (push tests required for other cases)

# Annex G: Design model for open trough profiles




$$P_{Rd} = \frac{0{,}58\, f_u\, \pi\, d^2/4}{\gamma_V} \qquad \text{(G.1)}$$

$$P_{Rd} = \frac{k_{cc}\, C_2\, k_u}{\gamma_V}\left[\frac{f_{ctk,0,05}\, W_{sc}}{h_p\, n_r} + \frac{n_y\, M_{pl,sc}}{\left(0{,}82\, h_p - d/2\right)}\right] \qquad \text{(G.2)}$$

$$C_2 = 1{,}85\frac{h_p}{b_0} \quad \text{but} \quad 1{,}0 \le C_2 \le 1{,}35 \qquad \text{(G.3)}$$

$$W_{sc} = \left[2{,}4\, h_{sc} + \left(n_r - 1\right)s_y\right]\left(\frac{b_{top}^2}{6}\right) \qquad \text{(G.4)}$$

$n_y = 2$ for $n_r = 1$ — for studs in a staggered position, see Figure G.1; (G.5)

$$n_y = 1 + \frac{h_A - 2\, d}{0{,}52\, d} \le 2 \quad \text{for other cases} \qquad \text{(G.6)}$$

$$M_{pl,sc} = \frac{1}{6} f_u\, d^3 \qquad \text{(G.7)}$$

# Annex J: Composite beams with precast concrete slabs

- A high proportion of multi-storey steel frames use precast concrete floors, which are particularly suited to sectors such as hotels, residential buildings and car parks.
- Whilst rules on precast slabs were given in Eurocode No. 4, ENV 1994-1-1 and ENV 1994-2, this information is not included in the current version of Eurocode 4.
- In the absence of design rules in Eurocode 4, design guides have been published in some countries which have been successfully used for more than 20-years.
- Generic design rules presented within Annex J

Source : Hicks SJ, Lawson RM (2003), Design of composite beams using precast concrete slabs. SCI Publication 287, Steel Construction Institute, Ascot, p. 92

# Support for second generation of EN 1994-1-1

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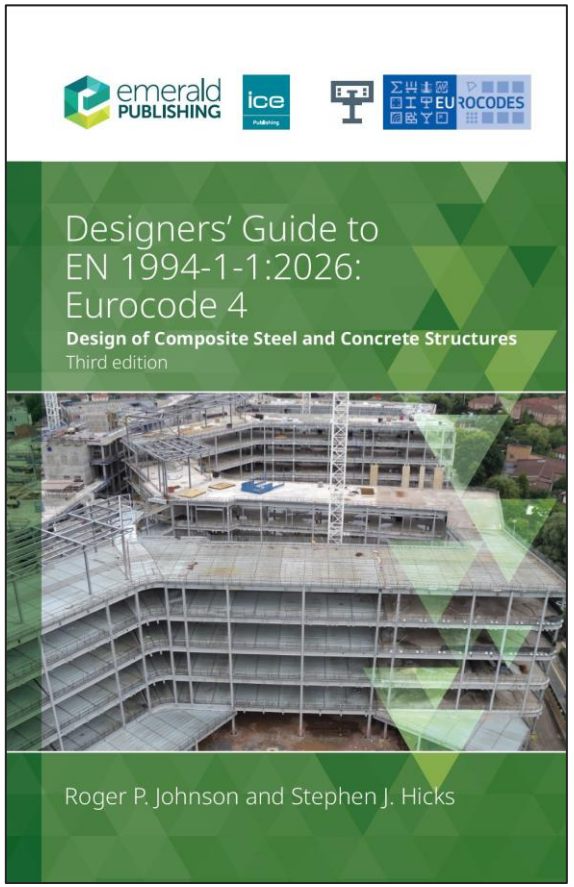


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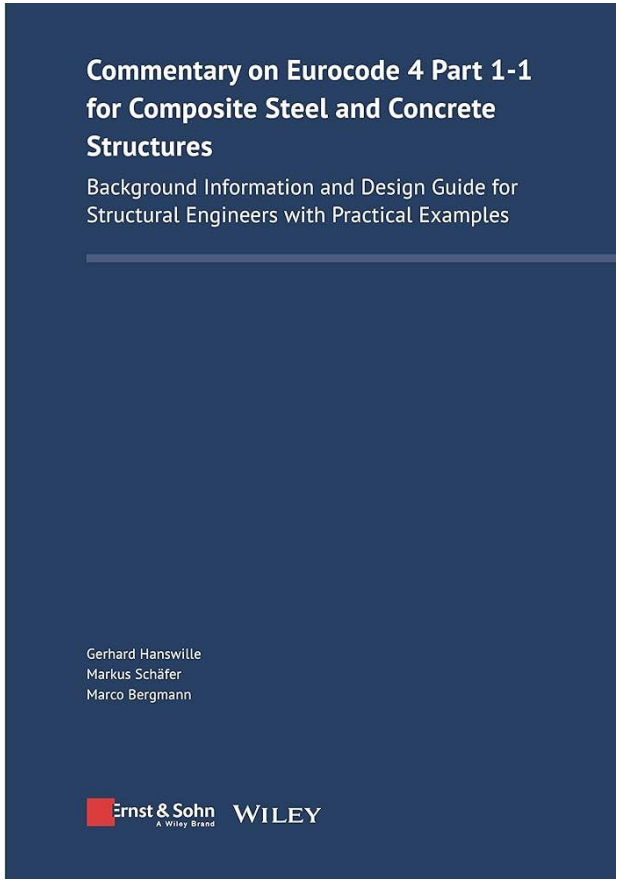
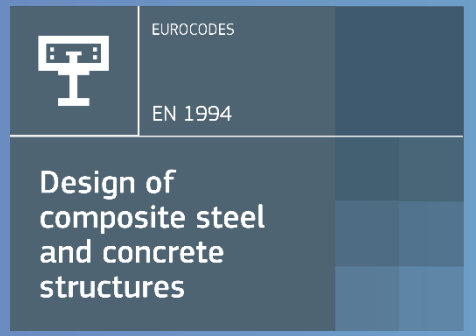
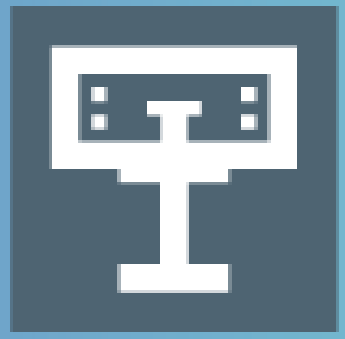


# EN 1994 ‘Eurocode 4: Design of composite steel and concrete structures’

Stephen Hicks, Mike Banfi, Wolfgang Kurz, François Hanus; CEN/TC 250/SC 4



# Items covered in this presentation

Web openings

Strain limited design

Limitations on the use of partial interaction

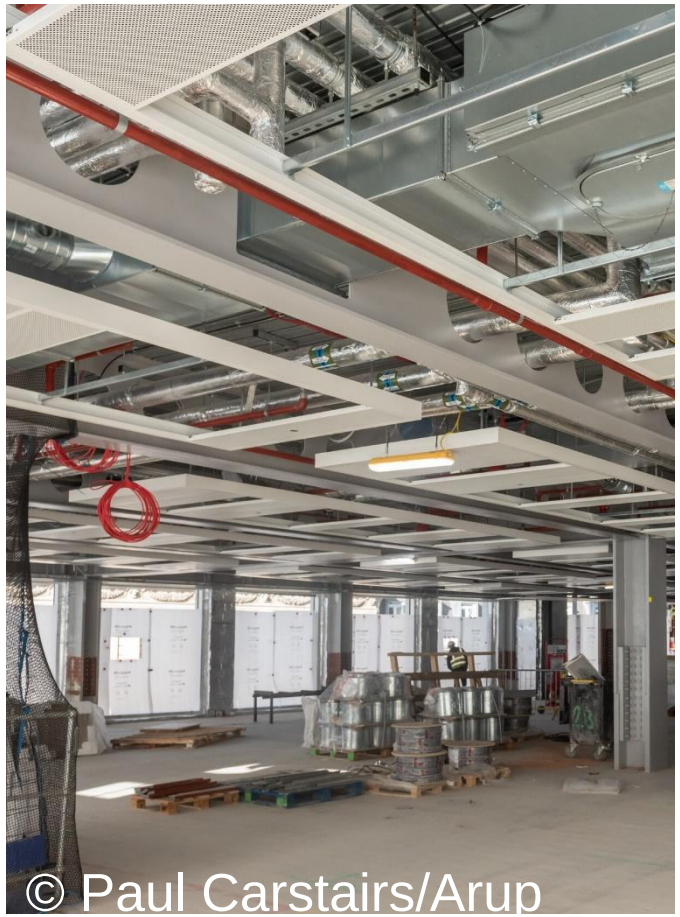
Shallow floor beams

# Web-openings in composite beams

Guidance on the use of beams web openings has been available in many countries

Under the European M515 project a team developed rules for web openings in steel beams (EN 1993-1-13) and the additional rules for composite beams.

The additional rules are limited to openings in sagging moment regions



# Annex D and Annex E

Annex D is normative and deals with composite beams where the flexural stiffness of the concrete slab is not significant and is not considered in the verification of the beam.

Where the flexural resistance of the slab is significant there are additional rules in Annex E. This is informative.

This presentation deals with the rules in Annex D. Those in Annex E will be covered in another presentation.

# Items covered

- Where simple rules may be applied
- The distribution of axial force from the global moment
- The forces in the web-post between openings
- Shear in the concrete slab
- Vierendeel bending resistance
- Serviceability

# Simple rules - clause D.2.2

EN 1993-1-13  reduces the number of checks that need to be applied for openings within certain limits. For such openings, a check on the global shear resistance can be sufficient, a check on global bending at the opening is not required.

For composite beams, the web has a more significant influence on the moment resistance so even for holes within the limits, EN 1994-1-1 D.1.2 requires that the check on global bending is carried out.

# Axial Force due to global moment (D.4.2)

The applied moment ($M_{o,Ed}$) is resisted by a couple between the tension in the bottom tee ($N_{bT,Ed}$) and compression in the slab ($N_{oc,Ed}$) and/or top tee ($N_{tT,Ed}$). There are three situations:

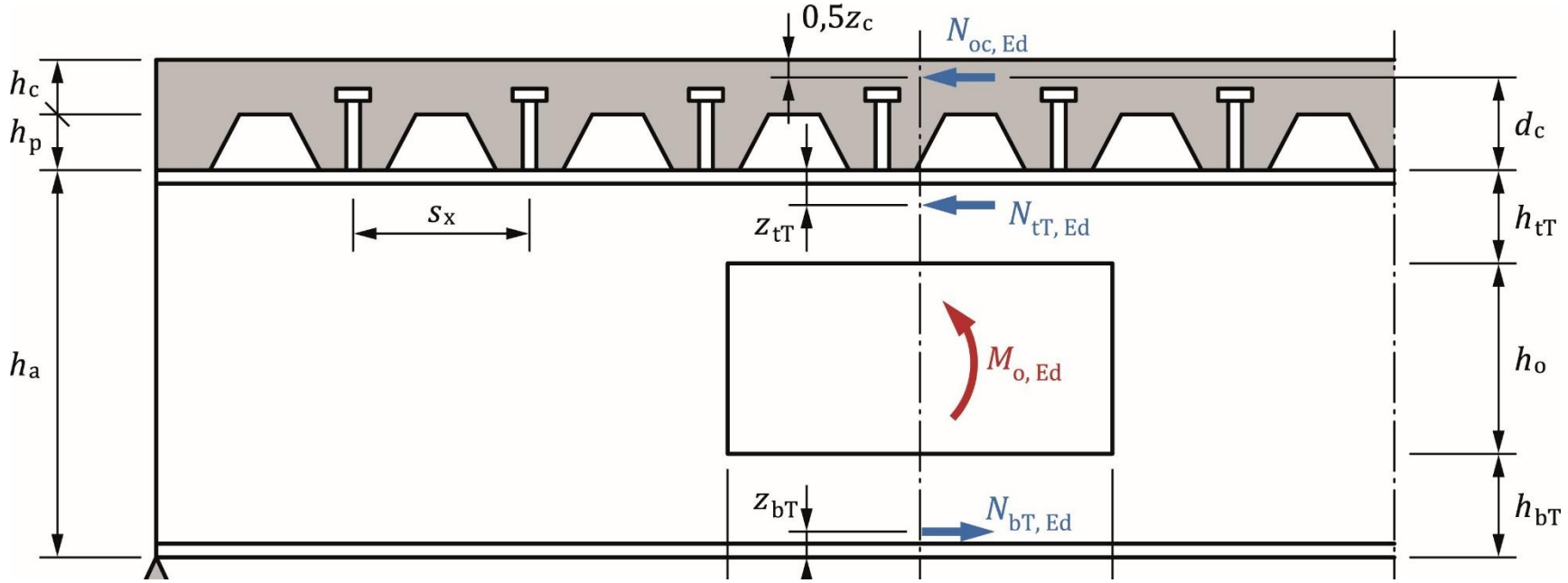



*Figure D.2 © BSI

# Axial force (D.3.2) – three situations

1. Both the force in the bottom tee and slab are less than the resistances:

$$N_{\mathrm{bT,Ed}} = N_{\mathrm{oc,Ed}} = \frac{M_{\mathrm{o,Ed}}}{h_{\mathrm{a}}+d_{\mathrm{c}}-z_{\mathrm{bT}}} \text{ and } N_{\mathrm{bT,Ed}} \leq N_{\mathrm{bT,Rd}} \text{ and } N_{\mathrm{oc,Ed}} \leq N_{\mathrm{oc,Rd}}$$

$$N_{tT,Ed} = 0$$

2. The force in the slab is limited by the resistance:

$$N_{\mathrm{oc,Ed}} = N_{\mathrm{oc,Rd}} \quad N_{\mathrm{bT,Ed}} = \frac{M_{\mathrm{o,Ed}} - N_{\mathrm{oc,Ed}}(z_{\mathrm{tT}} + d_{\mathrm{c}})}{h_{\mathrm{a}} - z_{\mathrm{tT}} - z_{\mathrm{bT}}} \quad N_{\mathrm{tT,Ed}} = N_{\mathrm{bT,Ed}} - N_{\mathrm{oc,Ed}}$$

3. The force in the bottom is limited by the resistance:

$$N_{\mathrm{bT,Ed}} = N_{\mathrm{bT,Rd}} \quad N_{\mathrm{tT,Ed}} = \frac{N_{\mathrm{bT,Ed}}(h_{\mathrm{a}} + d_{\mathrm{c}} - z_{\mathrm{bT}}) - M_{oEd}}{z_{\mathrm{tT}} + d_{\mathrm{c}}}$$

$$N_{\mathrm{oc,Ed}} = N_{\mathrm{bT,Ed}} - N_{\mathrm{tT,Ed}}$$

# Axial forces – resistance (D.5.1.2)

The axial forces in the steel tees above and below the opening are limited by the yield of the steel section provided the section is not slender.

The axial force in the concrete slab is limited by the axial resistance of the concrete and also by the force that can be transmitted by the shear connectors between the hole and the end of the beam

# Web-post forces (D.4.4)

The forces in the web posts are calculated from equilibrium

In common with EN 1993-1-13, the distribution of shear between the top and bottom tee can be chosen such as to minimise the web-post moment.

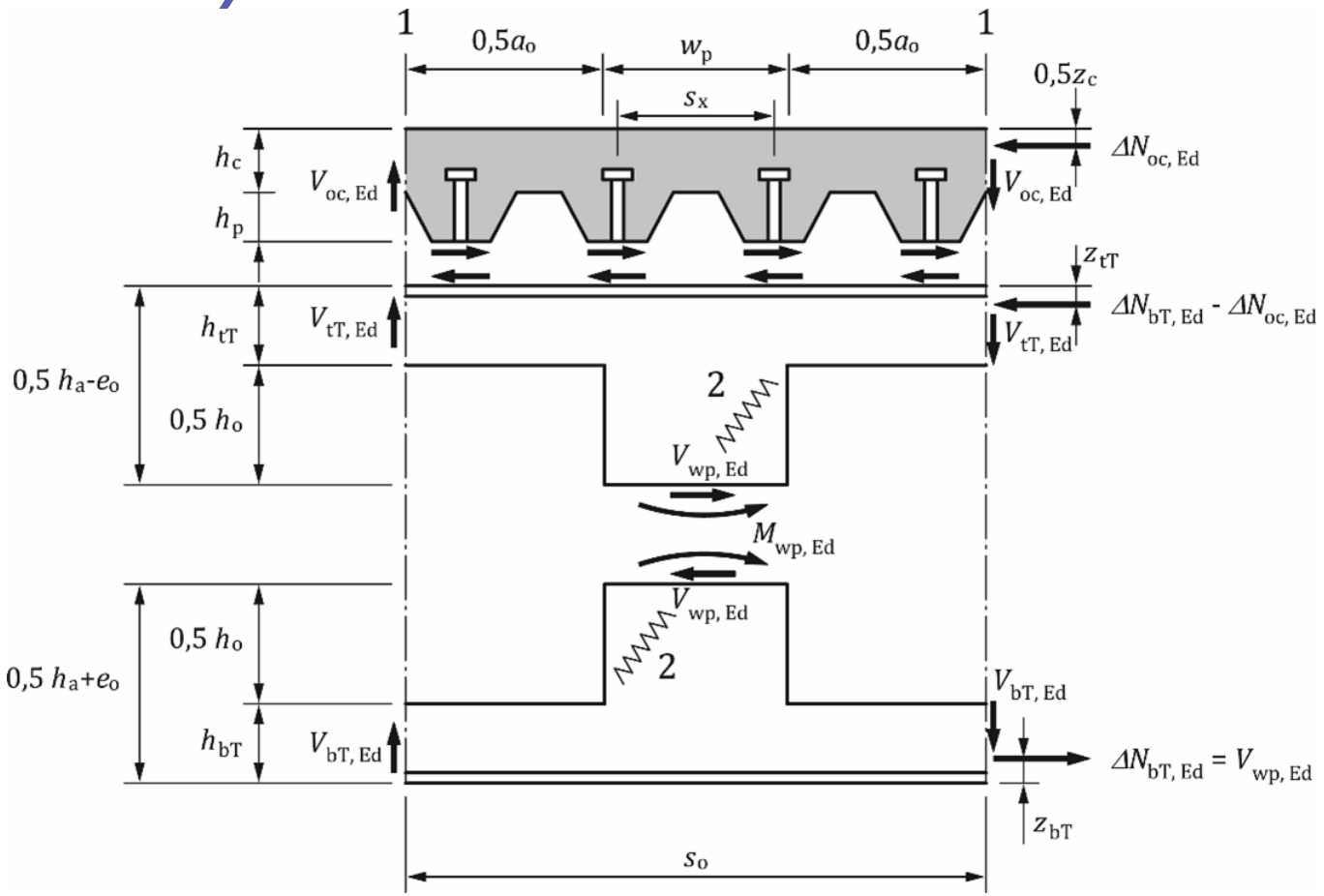
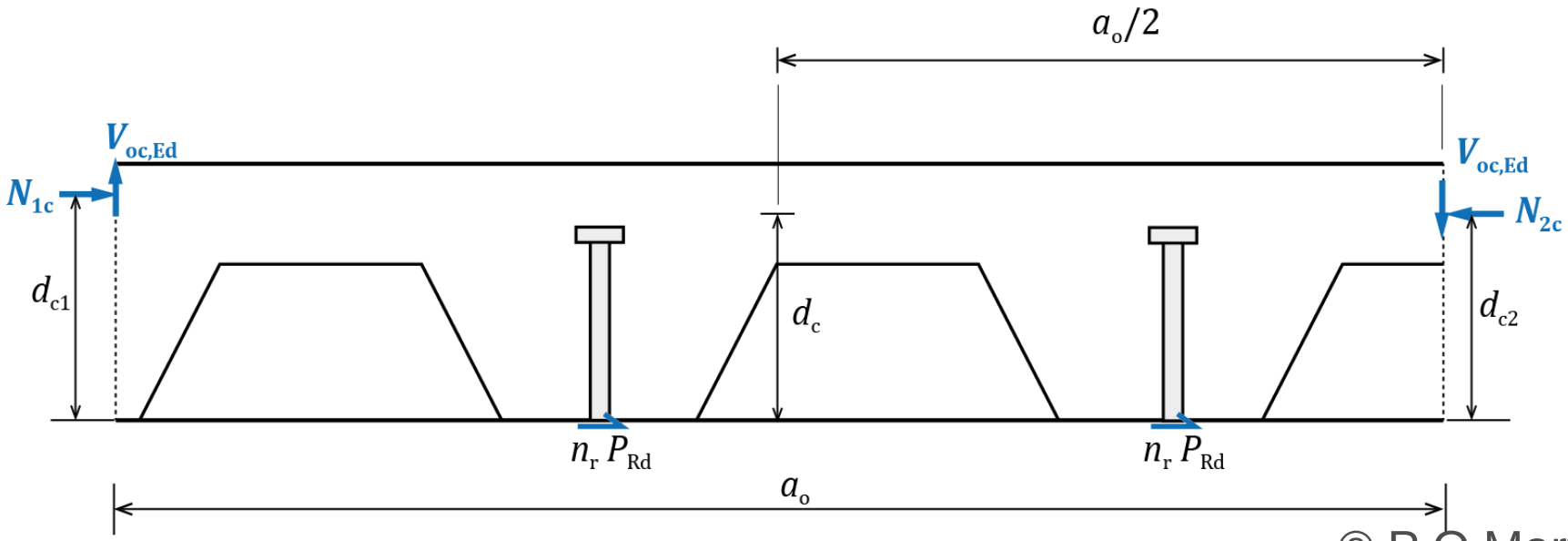



*Figure D.4 © BSI

# Shear in the concrete slab D.5.1.3

The rules in Annex D do not assume there are large shear forces in the concrete slab. However, there will be some shear due to the forces in the cross section as shown below.



# Shear in the concrete slab D.5.1.3

Equilibrium of the concrete slab leads to the expression for the shear in the concrete.

$$V_{\mathrm{oc,Rd}} = \frac{n_{\mathrm{r}}\, P_{\mathrm{Rd}}}{s_{\mathrm{x}}} \left( d_{\mathrm{c}} - \frac{1}{2} \frac{N_{\mathrm{oc,Ed}}}{b_{\mathrm{eff}}\, f_{\mathrm{cd}}} \right)$$

This is formulae D.28 with the additional limit that the shear cannot be more that the resistance of the concrete calculated according to EN 1992-1-1

Note $b_{\mathrm{eff}}$ is the effective width of the concrete slab and $f_{\mathrm{cd}}$ is the design strength of the concrete. Other symbols are as shown in previous figures in this presentation.

# Contribution to Vierendeel bending (D.5.1.5)

Although the flexural stiffness of the slab is neglected in Annex D the concrete can still contribute to the Vierendeel resistance through composite action between the top tee and the slab. Consideration of equilibrium of the top tee and slab leads to formulae D.31.

$$M_{\mathrm{vc,Rd}} = \left( V_{\mathrm{oc,Rd}} + \frac{n_{\mathrm{r}}\, P_{\mathrm{Rd}}}{s_{\mathrm{x}}} z_{tT} \right) a_{eq}$$

Note 1 $a_{\mathrm{eq}}$ is the equivalent length of the opening as defined in EN 1993-1-13. Other symbols are as shown in previous figures in this presentation.

Note 2 Where the spacing of shear connectors exceeds 0,5 time the length of the opening this contribution cannot be used.

# Contribution to Vierendeel bending (D.5.1.5)

A reduction factor ($k_a$) must be applied to this moment to take account of deformation across the opening and tensile forces in shear connectors.

The value of $k_a$ is given in Formula D.32 for unstiffened openings and D.33 for stiffened openings.

For un-stiffened where $a_{\mathrm{eff}} > 5h_{\mathrm{tT}}$ $k_{\mathrm{a}} = \left(1 - \frac{a_{\mathrm{eff}}}{25h_{\mathrm{tT}}}\right)$

For stiffened where $a_{\mathrm{eff}} > 7h_{\mathrm{tT}}$ $k_{\mathrm{a}} = \left(1 - \frac{a_{\mathrm{eff}}}{35h_{\mathrm{tT}}}\right)$

# Serviceability (D.6)

Web openings effect the deflection of the beam due to the reduction in flexural stiffness at the opening and the additional deflection across the opening due to Vierendeel bending.

In D,6 there are expressions for the additional deflection of uniformly loaded beams due to a widely spaced opening. These are based on elastic properties.

There are also expressions for the additional deflection of composite beams with regularly spaced circular of hexagonal openings under uniform loading. These are based on studies of a range of beams and are limited by the scope of the study.

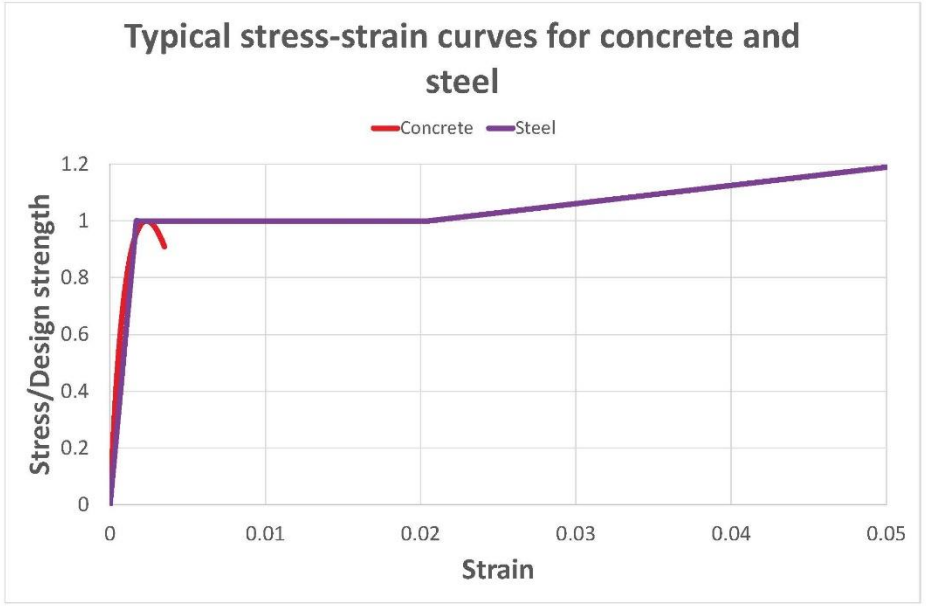
# Strain limited design

Traditionally composite beams are design plastically.

This assumes that the section can deform significantly.

The limiting strain for concrete is much less than it is for steel.

For typical beams the neutral axis is high and the concrete strain limit does not affect the resistance.



# Strain limited design – moment resistance (8.2.1.2)

When the neutral axis is lower the concrete may reach its limiting strain before the steel section has completely yielded.

Based on studies of numerous beams, a simple reduction factor on the moment resistance is given.

S420 and S460 as first generation

There is also a limit on the depth of the neutral axis for plastic design

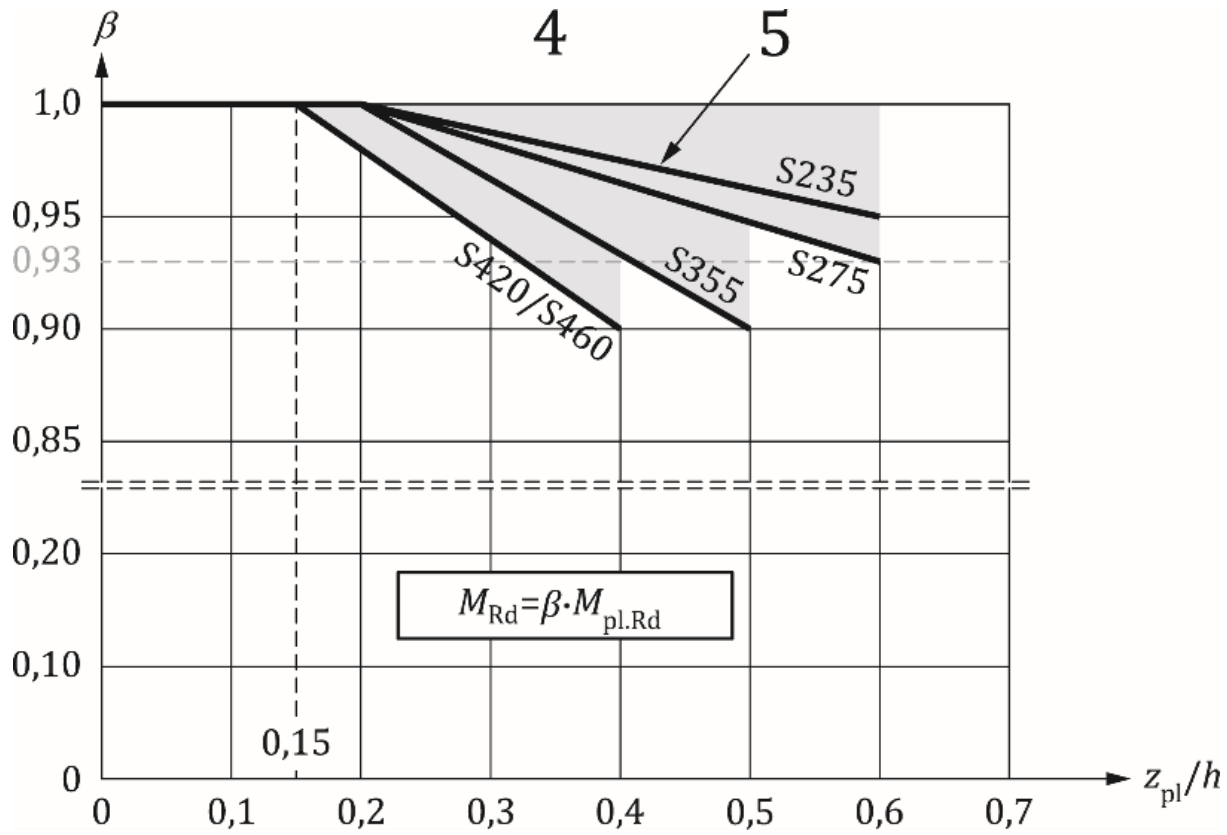



*Figure 8.3 © BSI

# Strain limited design – simple rule (8.2.1.2)

No reduction in moment resistance is necessary where all the following apply:

- The steel section is a typical doubly symmetric rolled section
- The steel grade does not exceed S355
- The depth of the steel is not less that 2.5 times the overall slab depth
- The plastic neutral axis is not below the top steel flange
- The slab is either solid or there is a minimum thickness of concrete above the profiled sheeting

# Strain limited design – with partial interaction (8.2.1.3)

When there is partial interaction the reduction in moment resistance can be taken as being proportional to the amount of partial interaction

This applies for beams with symmetrical steel sections or with asymmetrical steel sections and a neutral axis for full interaction not below the top flange.

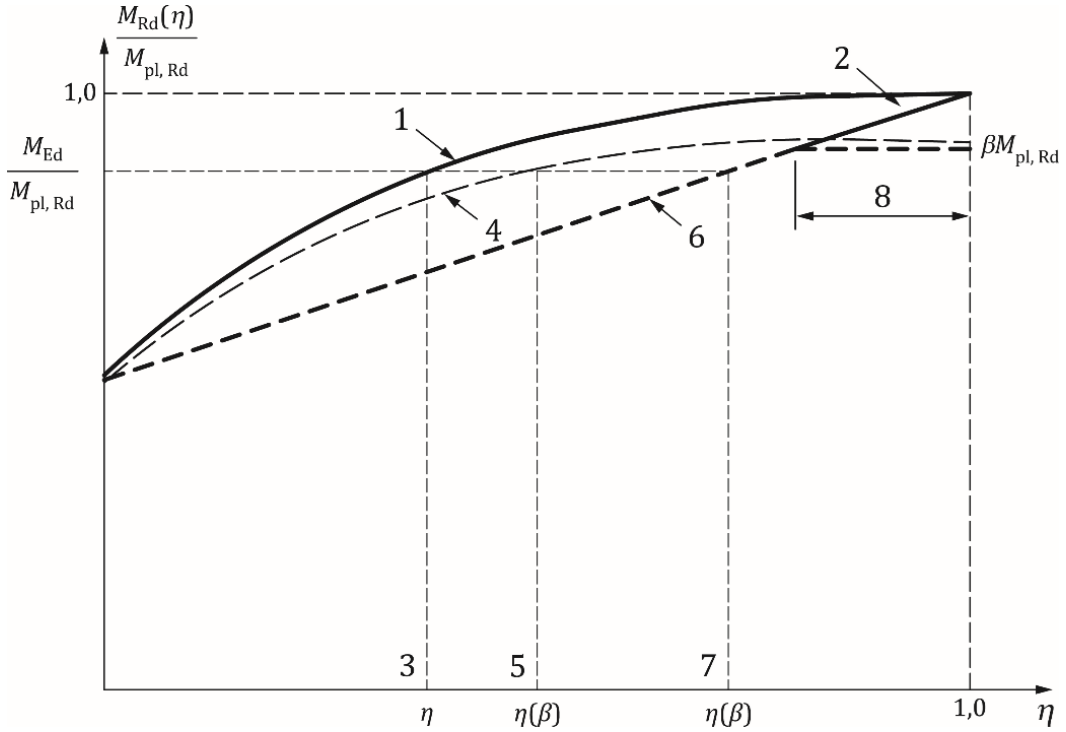



*Figure 8.6 © BSI

# Strain limited design – horizontal shear (8.6.2.3)

The plasticity in bending of typical composite beams helps to allow a uniform distribution of shear connectors.

If the neutral axis is within the limits in Figure 8.3 a uniform distribution is usually OK although additional checks can be required.

If the neutral axis is outside the limits in Figure 8.3 more advanced methods of calculation are required for the distribution of shear connection.

# Limitation on the use of partial shear connection (8.6.3.3)

The limitation on the use of partial interaction is to prevent the slip of the shear connectors exceeding the capacity.

The rules in the 2004 version of EN 1994-1-1 were based on studies of a limited number of composite beams. They were based on a deformation capacity of 6mm and assumed that the beam was constructed unpropped and was working a 95% of its resistance.

The 2nd generation version of EN 1994-1-1 is explicit in saying that the verification of the amount of partial interaction can be demonstrated by showing that the calculated slip is less than the capacity of the connectors. This would require advanced methods of analysis. There are also simple rules similar to the 2004 version.

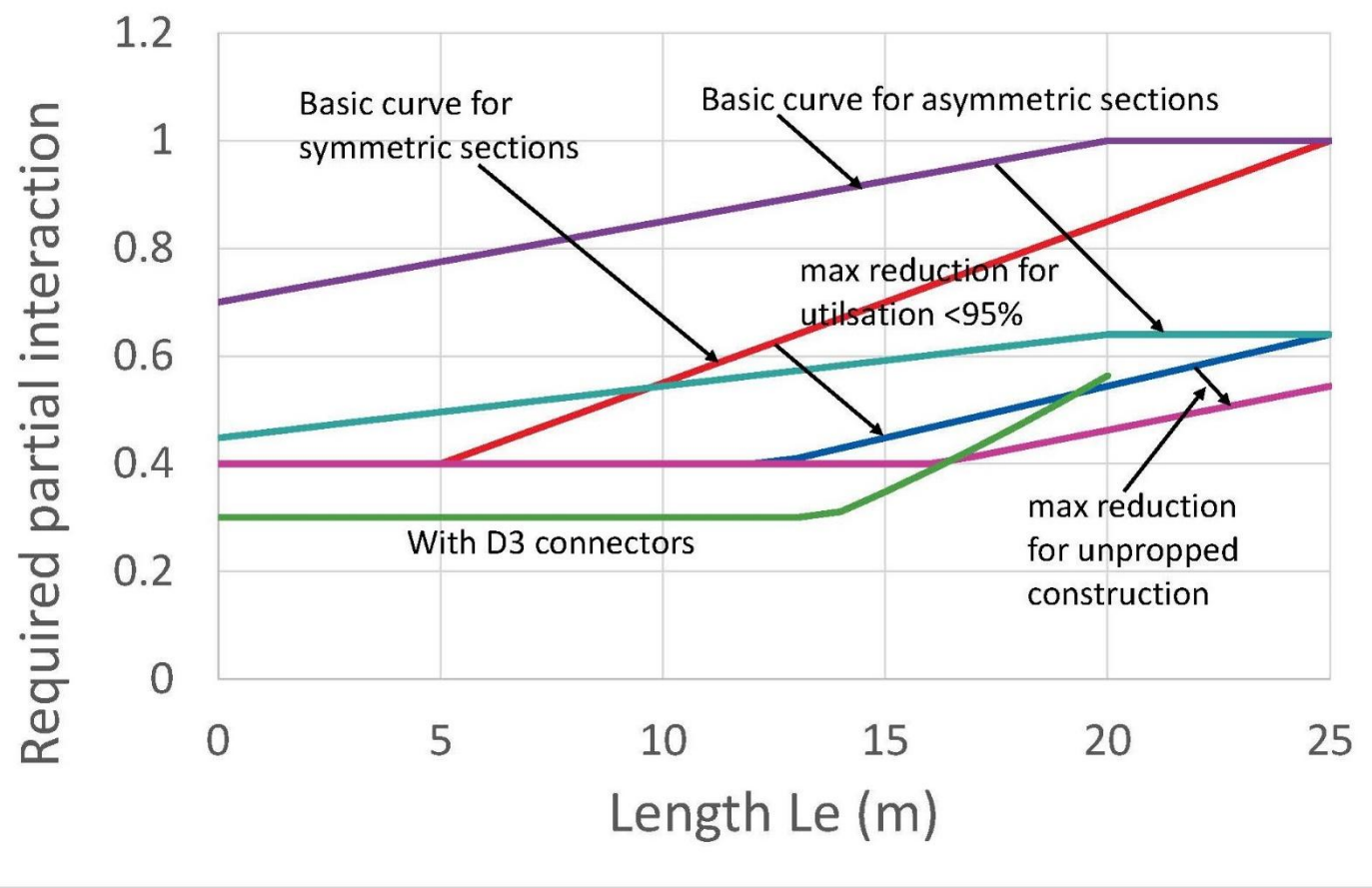
# Limitation on the use of partial shear connection (8.6.3.3) – simple rules

The basic equations for partial interaction are unchanged.

However, there are modifications for:

- The applied moment being less than 95% of the moment resistance
- The beam being unpropped during construction
- The shear connectors being Ductility Category D3 with a deformation capacity of at least 10mm.

# Limitation on the use of partial shear connection (8.6.3.3) – simple rules

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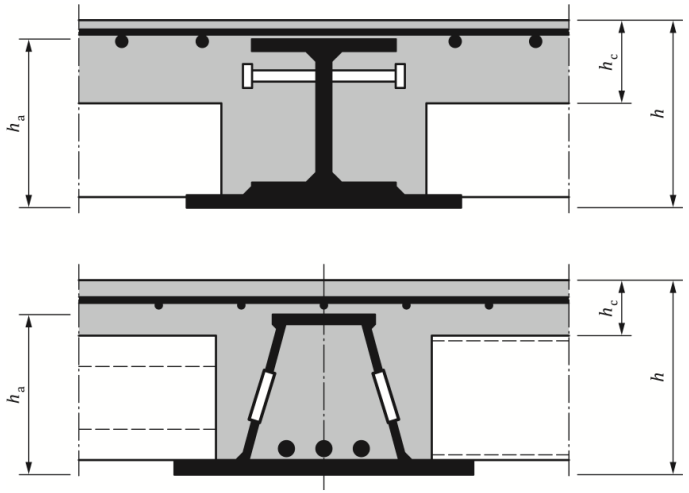
# Limitation on the use of partial shear connection (8.6.3.3) – simple rules

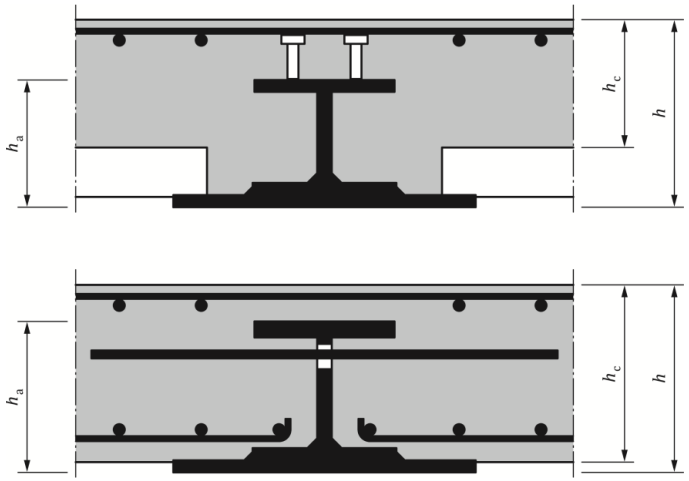
For deeper beams, where the distance between the centroid of the steel beam and the centroid of the effective concrete slab exceeds 600mm the amount of partial interaction needs to be increased.

# Shallow floor beams – Annex I

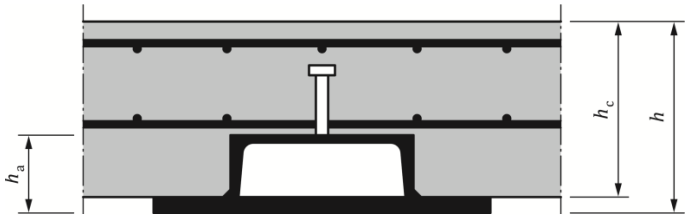
- Divided into types depending on the contribution of the slab to the flexural stiffness of the beam




- Type I
- Not significant




- Type II
- Significant




- Type III
- Significant

*Table I.1 © BSI

# Shallow floor beams – Annex I

The amount of moment redistribution for type I beams is limited. For type II and III the rules in EN 1992 must be used.

Additional rules given for classification of steel elements as these can be affected by the concrete

The transverse bending of plates e.g. the bottom flange supporting precast slabs, can affect the longitudinal strength. A simple rule is given.

Simple rules for partial interaction are given for beams where the bottom flange is bigger than 3 times the top flange.

# Shallow floor beams – Annex I
# Vertical shear

Rules given for the possible contribution of the concrete to the shear resistance

For type II and III beams the concrete must be able to resist a minimum level of shear.

Where the end connection is steel to steel the applied shear should not exceed the shear resistance of the steel section.

# Shallow floor beams – Annex I
# SLS

Where the flexural stiffness of the slab is significant e.g. Type II and III beams, the normal rules for the calculation of composite properties can underestimate the flexural stiffness.

Methods to give more accurate value of the second moment of inertia of type II and type iii beams are given.

# EN 1994 ‘Eurocode 4: Design of composite steel and concrete structures’

Stephen Hicks, Mike Banfi, Wolfgang Kurz, François Hanus; CEN/TC 250/SC 4

EUROCODES

EUROCODES

EN 1994

Design of composite steel and concrete structures

# Web-openings in composite beams
# Specific rules in Annex E

Annex E takes into account the additional stiffness and shear capacity of the slab above the opening.

This allows for additional Vierendeel bending resistance and provides additional design checks against vertical shear in the slab and tension failure of headed studs.

# Criteria for stiff slabs in Annex E

A slab should be considered to be locally stiff when both following conditions apply:

- $h_c \geq h_{tT}$
- longitudinal reinforcement ratio ρ greater than 0,3% within the effective width $b_{eff}$

For beams with regular closely spaced web openings slabs may generally be assumed to be locally flexible.

# Effective width of the slab (E.2)

The effective width $b_{eff,b}$ for local bending resistance is connected to the opening length $a_0$ and the local width of the shear connection $d_y$:

$$b_{eff,b} = d_y + 0{,}83a_0$$

For a single row of shear connectors $d_y$ may be assumed as 7,5 $\Phi_t$ which is the diameter of the transverse reinforcing bars.

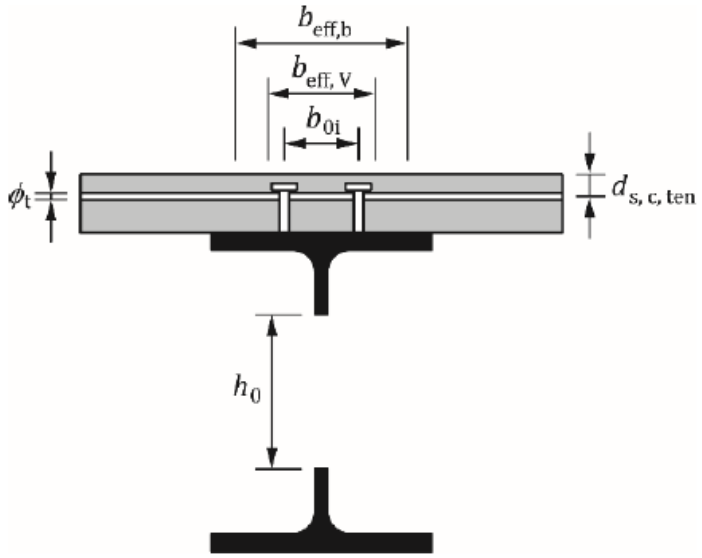



*Figure E.1 © BSI

# Bending in the slab

- Flexural stiffness leads to significant bending moments in the slab $M_{A,Ed}$ , $M_{B,Ed}$
- Local tension forces at the high moment end occur, also compression forces at the lower moment edge
- Tension has to be covered by connectors
- Shear in the concrete slab has to be checked
- Load introduction length is taken as the length of the opening

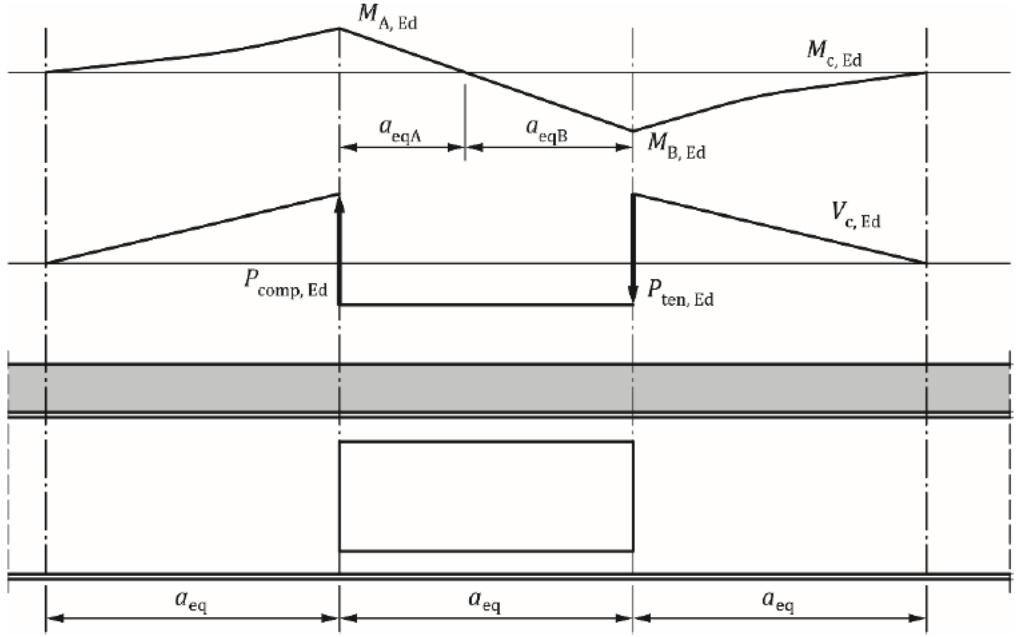



*Figure E.2 © BSI

# Shear forces in the slab

The vertical shear force in the slab at the centre of the opening shall be determined by:

$$V_{slab} = V_{o,Ed} - \frac{2\left(M_{elN,tT} - M_{elN,bt}\right)}{a_{eq}} - n_r P_{Rd} \frac{d_c}{s_x}$$

Here $M_{elN,tT}$ is the bending moment in the top steel Tee causing yielding at the high moment end of the opening and $M_{elN,bT}$ is the bending moment at the low moment end of opening in the bottom Tee, both taking into account the axial forces. $d_c$ is taken as the effective height $h_p + 0{,}5h_c$ and $s_x$ is the longitudinal distance of shear connectors.

Alternatively the shear in the slab may be determined by checking the construction sequence.

# Actions in the slab due to vertical shear (E.4)

Vertical shear leads to bending moments at the ends of the opening:

$$M_{A,Ed} = V_{slab} a_{eq} \frac{M_{A,Rd}}{M_{A,Ed}+M_{B,Rd}} \leq M_{A,Rd}$$

$$M_{B,Ed} = V_{slab} a_{eq} \frac{M_{B,Rd}}{M_{A,Ed}+M_{B,Rd}} \leq M_{B,Rd}$$

The vertical forces at the end of the opening may be obtained by:

$$P_{comp,Ed} = 2\left|M_{A,Ed}\right| \frac{a_{eqA}+0{,}5a_{eq}}{a_{eqA}a_{eq}} \qquad a_{eqA} = a_{eq} - a_{eqB} = a_{eq} \frac{\left|M_{A,Ed}\right|}{\left|M_{A,Ed}\right|+\left|M_{B,Ed}\right|}$$

$$P_{ten,Ed} = 2\left|M_{B,Ed}\right| \frac{a_{eqB}+0{,}5a_{eq}}{a_{egB}a_{eq}}$$

# Additional design checks (E.5)

1. Global bending resistance in accordance with Annex D and forces calculated by E.2

2. Global shear resistance in the slab with an applied shear force

$$V_{oc,Ed} = \frac{M_{A,Ed}+M_{B,Ed}}{a_{eq}} + n_r P_{Rd} \frac{d_c}{s_x}$$

3. Resistance to *Vierendeel* bending including the composite *Vierendeel* bending resistance

4. Resistance of connection at the opening edges

5. Web buckling (similar to Annex D)

6. Resistance of transverse reinforcement to local loads

# Resistance to shear forces (E.5.3+E.5.7)

The shear force in the concrete $V_{oc,Ed}$ should be less than the shear resistance given by EN 1992-1-1.

The difference between the total shear $V_{o,Ed}$ applied to the beam and the shear in the slab should be resisted by the steel Tee sections at the opening.

For the local load introduction of $V_{oc,Ed}$ a sufficient transverse reinforcement has to be provided:

$$\frac{V_{oc,Ed}}{F_{tr,Rd}} \leq 1{,}0 \qquad F_{tr,Rd} = \frac{\rho_t}{\rho_{min}} 0{,}006 h_c d_{s,c,b} f_{sd}$$

# Resistance to *Vierendeel* bending (E.5.4)

The shear resistance by *Vierendeel* bending at the opening may be considered as:

$$\frac{V_{o,Ed}}{V_{Vier,Rd}} \leq 1{,}0$$

$$M_{c,Rd} = M_{A,Ed} + M_{B,Ed}$$

$$M_{vc,Rd} = n_r P_{Rd}\left(a_{eq} - 0{,}5h_c\right)\left(\frac{d_c}{s_x} + \frac{z_{tT}}{s_x}\right)$$

$$V_{Vier,Rd} = \frac{2M_{NV,bT,Rd} + 2M_{NV,tT,Rd} + M_{vc,Rd} + M_{c,Rd}}{a_{eq}}$$

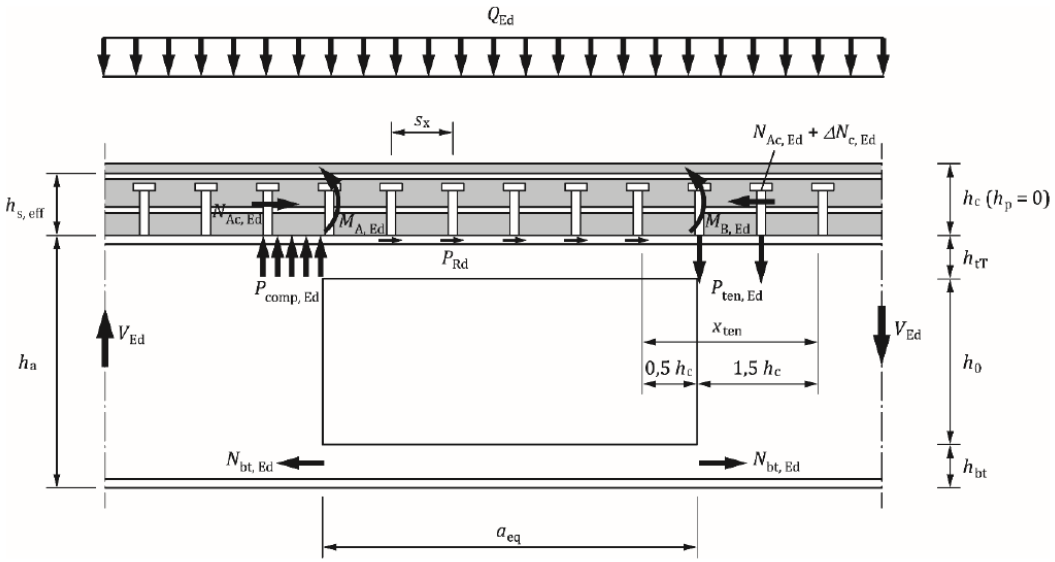



*Figure E.3 © BSI

# Resistance at the opening ends (E.5.5)

Resistance to compression force $P_{comp,Ed}$ is calculated as resistance against punching shear according to EN 1992-1-1

Resistance to tension is given by:

$$\frac{P_{ten,Ed}}{n_t P_{ten,Rd}} \leq 1{,}0$$

$n_t$ is the number of shear connectors in a length from $0{,}5(h_c + h_p)$ inside the opening to $1{,}5\ (h_c + h_p)$ outside the opening.

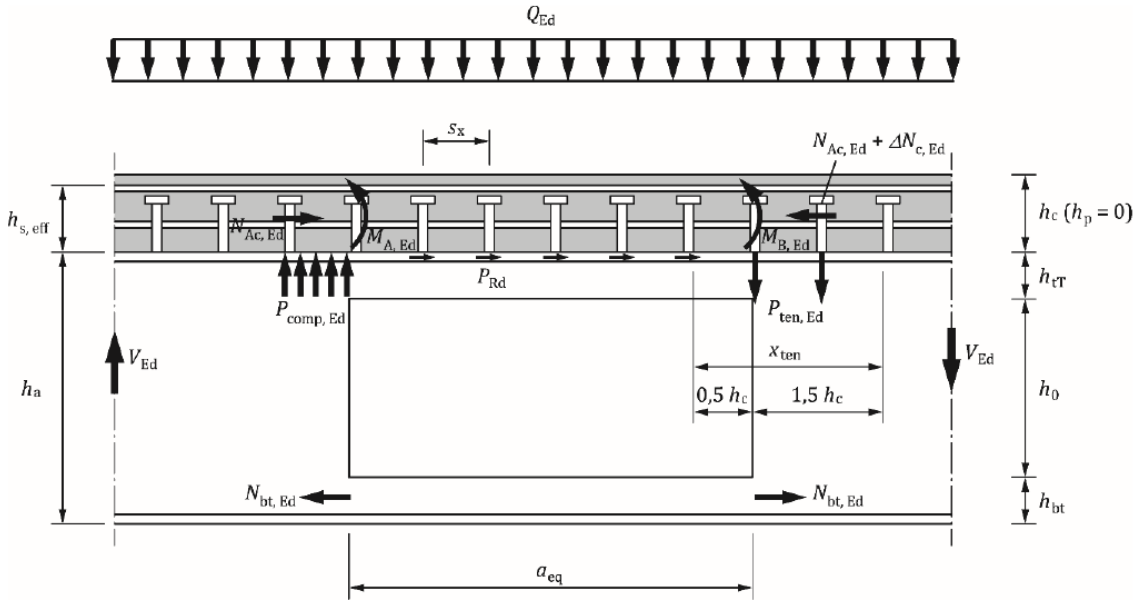



*Figure E.3 © BSI

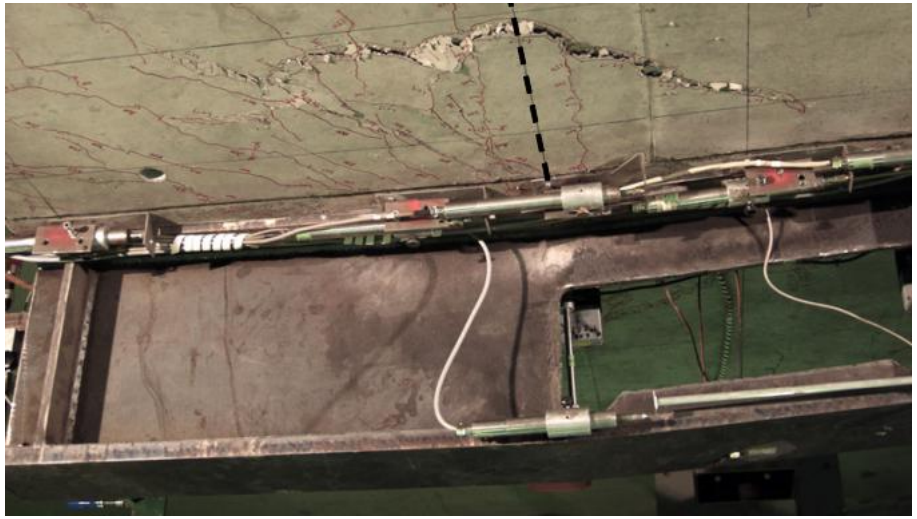
$$\frac{P_s}{P_{Rd}} + \frac{P_{ten,Ed}}{n_t P_{ten,Rd}} \leq 1{,}2$$ Interaction of shear and tension on headed studs

# Headed studs under tension loading (Annex H)

The first generation of EN 1994-1-1 did not provide rules for headed studs in tension. New applications as e.g. rules for beams with web openings provid e need for those rules.

Therefore EN 1994-1-1, Annex H provides rules derived from EN 1992-4 and from ETAs for common headed studs fulfilling the minimum requirements on material properties and taking into account possible failure modes.

$$P_{ten,Rd} = min\left(N_{s,Rd}; N_{p,Rd}; N_{c,Rd}\right)$$



© Christian Kohlmeyer

# Steel failure under tension (H.3.2(2))

The design resistance to steel failure is given by:

$$N_{s,Rd} = \frac{N_{s,Rk}}{\gamma_{Ms}}$$

| $d$ [mm] | 16 | 19 | 22 | 25 |
|---|---|---|---|---|
| $N_{s,Rk}$ [kN] | 90 | 128 | 171 | 221 |

*Table H.1© BSI

Note: Partial factor $\gamma_{Ms}$ is taken from EN 1992-4 but open to National choice.

# Pull-out failure under tension (H.3.2(3))

The design resistance depends on the stud shank diameter and the concrete strength:

$$N_{p,Rd} = \psi_p \frac{N_{p,Rk}}{\gamma_{Mp}}$$

| $d$ [mm] | 16 | 19 | 22 | 25 |
|---|---|---|---|---|
| $N_{p,Rk}$ [kN] | 90[a] | 75 | 85 | 115 |
| [a] the head diameter is assumed to be 32 mm | | | | |

*Table H.2© BSI

| Concrete Class | C25/30 | C30/37 | C35/45 | C40/50 | C45/55 | C50/60 |
|---|---|---|---|---|---|---|
| $\psi_p$ | 1,20 | 1,48 | 1,80 | 2,00 | 2,20 | 2,40 |

*Table H.3© BSI

# Concrete cone failure under tension (H.3.2(4))

The design resistance to concrete cone failure is given by

$$N_{c,Rd} = \frac{N_{c,Rk}}{\gamma_{Mc}}$$

$$N_{c,Rk} = N^{0}_{c,Rk} \frac{A_{c,N}}{A^{0}_{c,N}} \psi_{s,N} \psi_{re,N} \psi_{ec,N} \psi_{M,N}$$

$N^0_{c,Rk}$ ist the resistance for a single stud, $A_{c,N}/A^0_{c,N}$ takes into account geometric effects of stud spacing and edge distance. Ψ factors are for influences of slab edge, spalling and group effects by different loading.

# Concrete cone failure under tension (H.3.2(4))

Specific information are provided on the shape and size of concrete cones for solid slabs and composite slabs.

$$\frac{A_{c,N}}{A^0_{c,N}} = \frac{min\{b_x;3h_{ef}\}\times[3h_{ef}+(n_s-1)min\{s_y;3h_{ef}\}]}{9h^2_{ef}n_s}$$

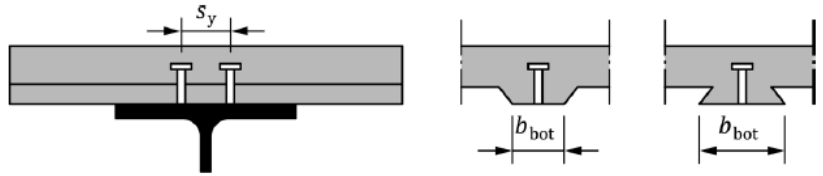



*Figure H.2© BSI

$$N^0_{c,Rk} = k_1\sqrt{f_{ck}}h^{1,5}_{ef}$$

Factor $k_1$ considers the load transfer mechanism and may be taken as 8,5 in cracked concrete.

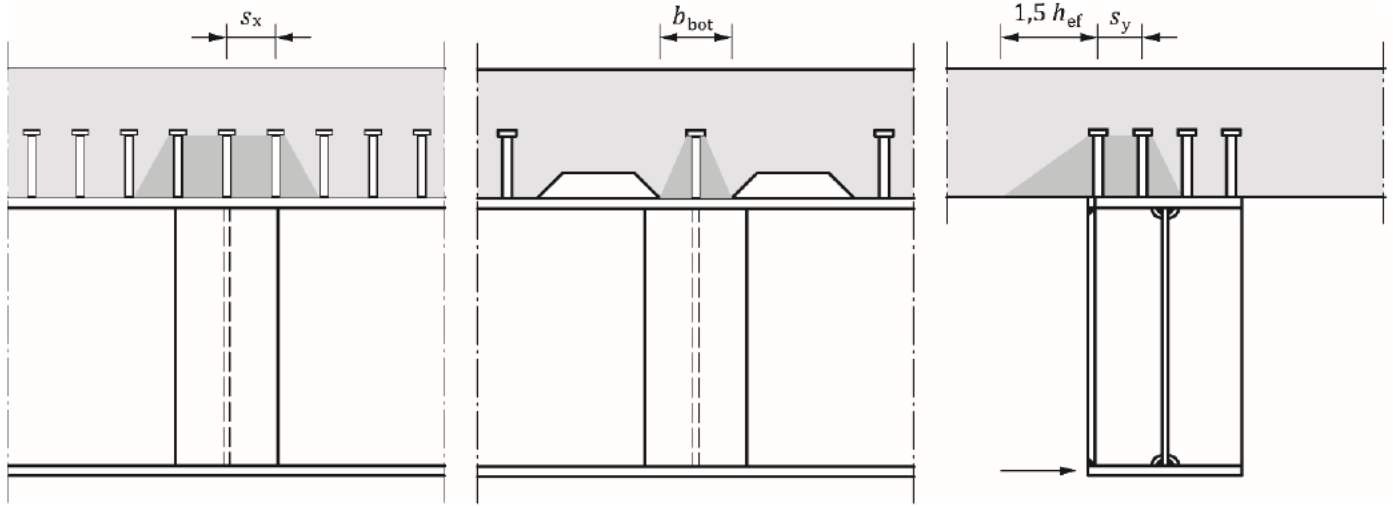



*Figure H.1© BSI

# Concrete cone failure (H.3.2(6..8))

ψ factors are determined in line with EN 1992-4 but specific geometric details are given for headed studs in composite slabs.

$\psi_{s,N} = 0{,}7 + 0{,}3\frac{c}{c_{cr,N}}$ Influence of edge distance

$\psi_{re,N} = 0{,}5 + \frac{h_{ef}}{200} \leq 1{,}0$ Influence of spalling

$\psi_{ec,N} = \frac{1}{1+2(e_N/s_{rc,N})} \leq 1{,}0$ Influence of load eccentricity

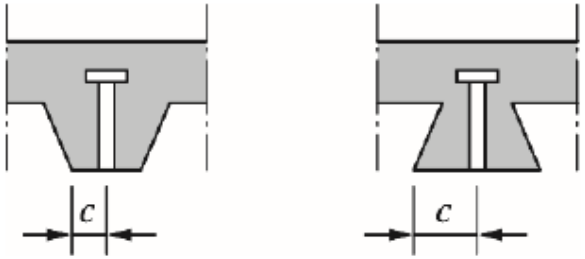



*Figure H.3© BSI

# General method for composite columns

In the first generation of Eurocodes the general method for design of composite columns was given in a very common way without specific information.

Second generation provides more details in clause 8.8.2.

The general method is based a non-linear analysis and takes account of non-linear effects including residual stresses, geometrical imperfections, Cracking of concrete, creep, shrinkage and yielding of both structural steel and reinforcement.

Internal forces shall be determined by a GMNIA analysis.

# General method (8.8.2)

The general method applies an overall factor $\gamma_0$ for resistance and action effects. Variations of material properties (e.g. elastic modulus) have to be taken into account if relevant.

$$N_{Ed}^{II} \leq \frac{R_m}{\gamma_0}$$

Here $R_m$ is the mean resistance obtained from GMNIA (failure load) and $N^{II}_{Ed}$ is the internal force determined from a non-linear analysis taking into account the combination of action effects in accordance with EN 1990 and EN 1991 and the partial factors $\gamma_G$ and $\gamma_Q$.

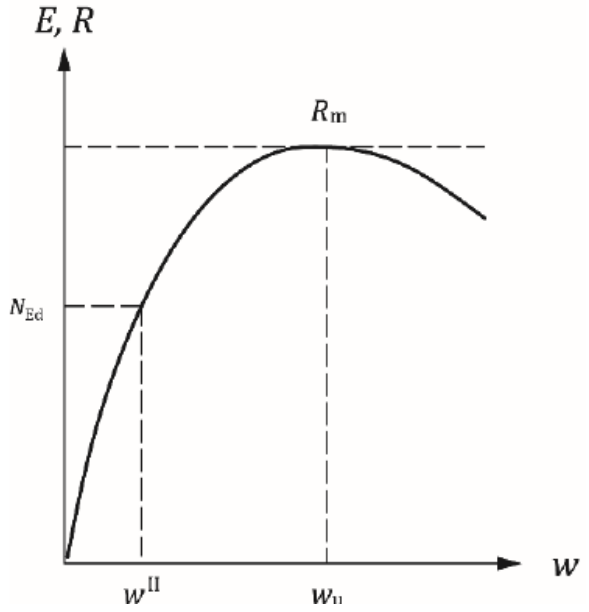



*Figure 8.25”1”© BSI

# General method (8.8.2)

The overall (partial) factor $\gamma_0$ depends on the N-M load vector. Therefore for each load combination a load vector $E^{II}_d$ has to be calculated.

Two N-M interaction curves have to be calculated for the cross section of the column. One is based on mean material properties and one on design properties of material.

For each load vector $E^{II}_d$ a specific factor $\gamma_0$ is obtained for verification of the column. For biaxial bending the factor $\gamma_0$ has to be developed for each bending axis.

Curves may be determined by plastic or strain-limited analysis.

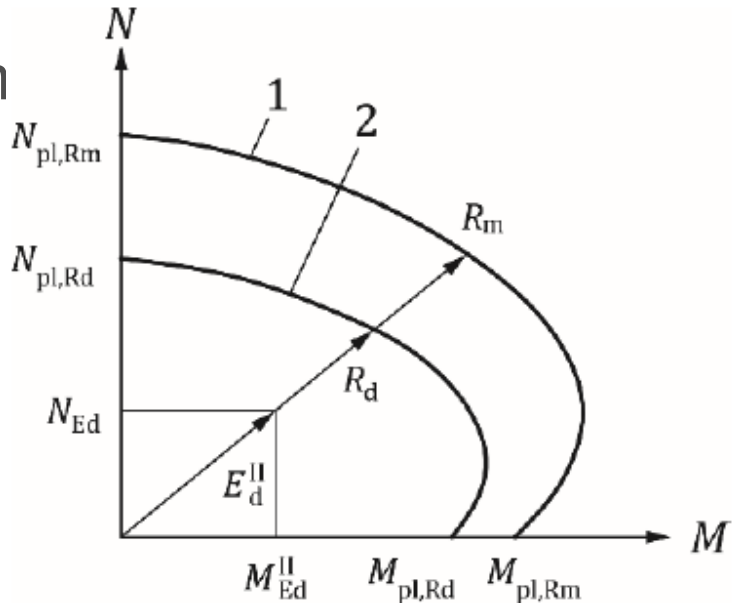
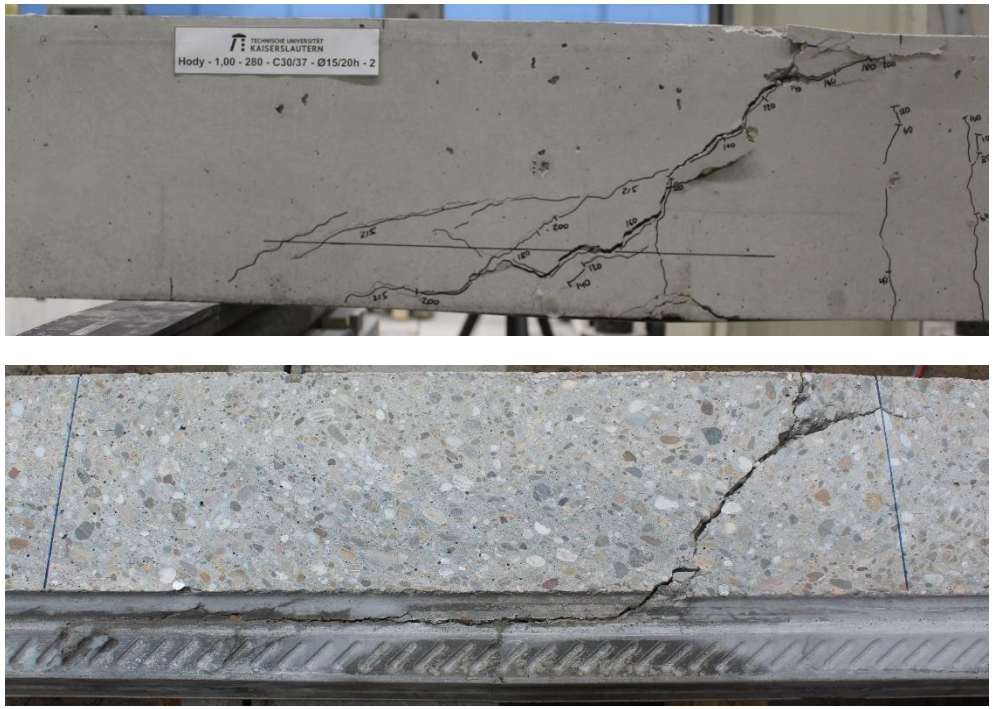



*Figure 8.26© BSI

# New design model for vertical shear (10.7.5)

First generation of Eurocode 4 did not provide specific design rules for vertical shear resistance in composite slabs which resulted in various design approaches.

Based on a research programm with various sheet profiles a design model for vertical shear in composite slabs has been developed and implemented.

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# New design model for vertical shear (10.7.5)

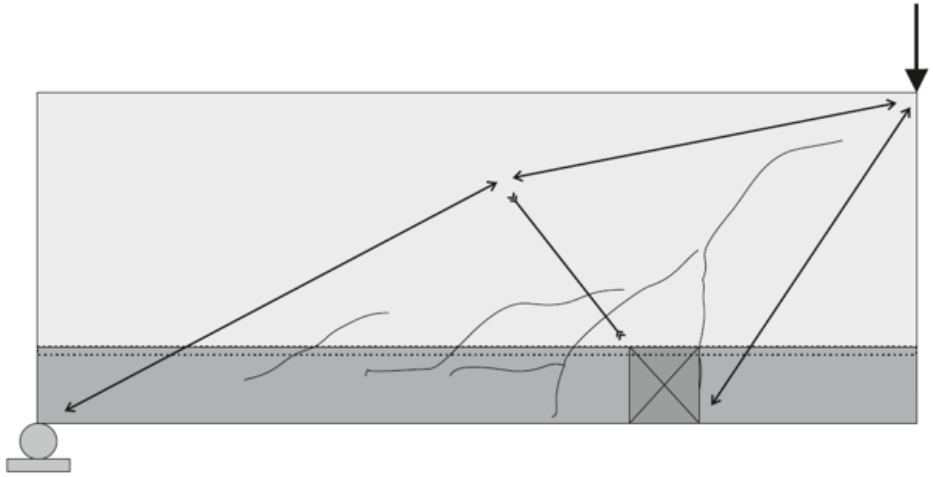
Design model is based on a truss model with four mechanisms transfering shear to the supports:

$V_{c,cz}$ shear resistance of the uncracked concrete compression zone

$V_{c,ks}$ shear resistance by dowel action and spalling of concrete

$V_{c,cs}$ shear resistance by a direct compression strut to the support

$V_{c,ct}$ shear resistance of concrete crack propagation zone



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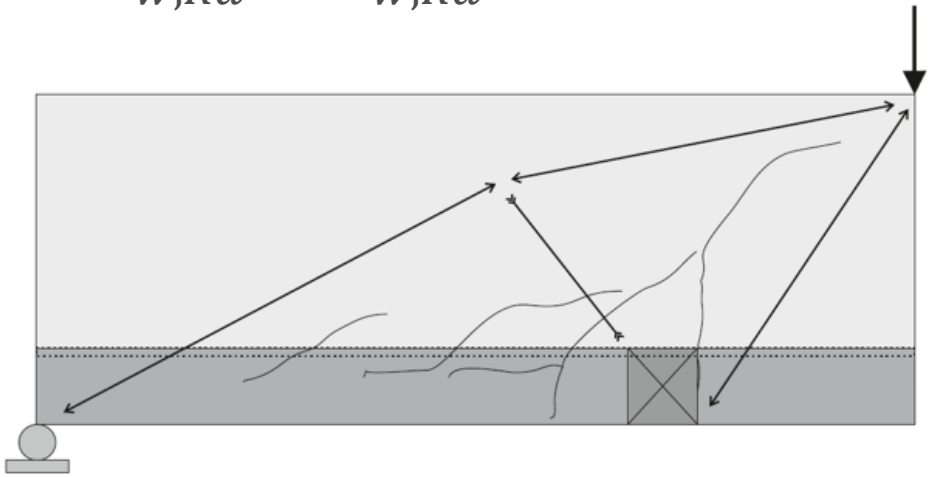
# New design model for vertical shear (10.7.5)

The shear resistance of the steel sheet itself also provides a contribution to the transfer of vertical shear. After calibration and statistical evaluation this leads to the new design equation:

$$V_{v,Rd} = \frac{0{,}5}{\gamma_c}\left(V_{c,cz} + V_{c,ks} + V_{c,cs} + V_{c,ct}\right) + 0{,}5R_{w,Rd} \geq R_{w,Rd}$$

$R_{w,Rd}$ is the design resistance to transverse force of the steel web that may be determined by testing or taken from ETAs.

This design model is also adapted for punching.



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# New design model for vertical shear (10.7.5)

EN 1994-1-1, 10.7.5 provides information for determination of each component of concrete contribution to vertical shear resistance and also information about the critical section for design checks.

Geometrical information depending on the shape of steel sheeting is also provided.

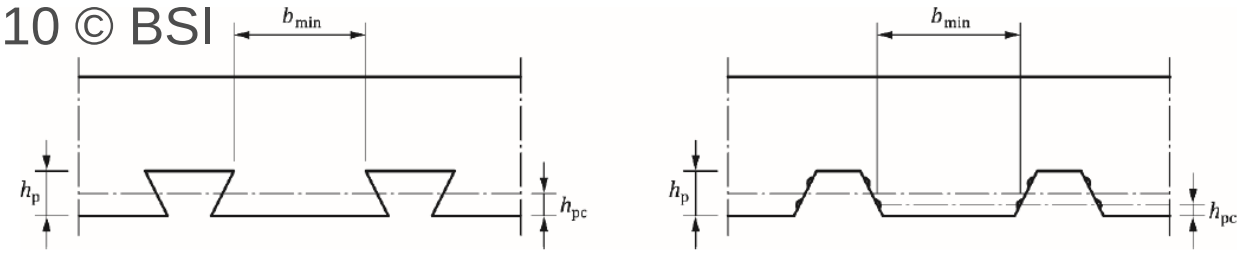



Figure 10.10 © BSI

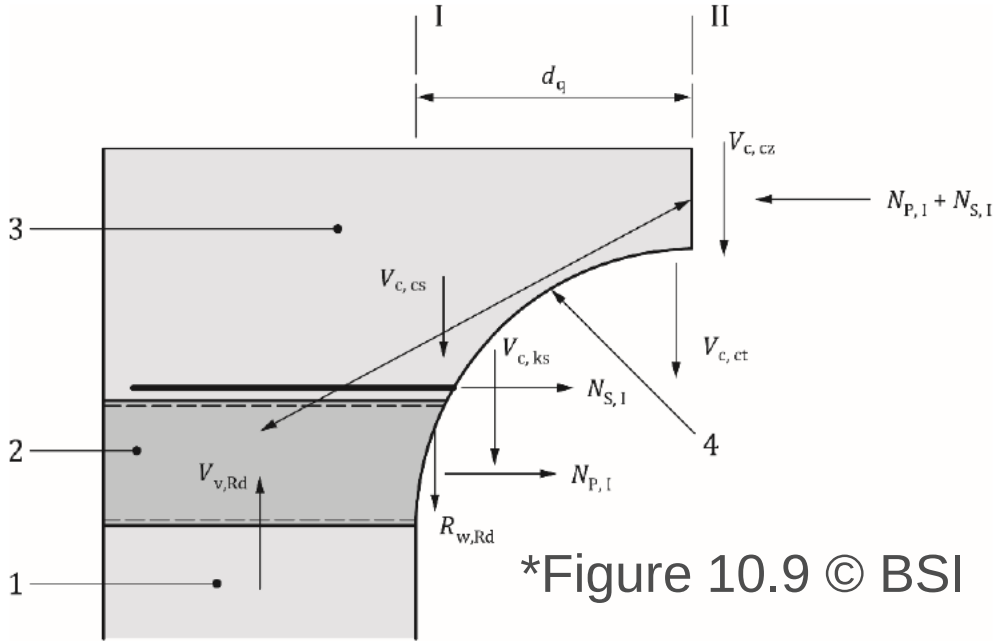



*Figure 10.9 © BSI

Based on National choice an alternative method may be applied.

# TS for composite dowels (CEN/TS 1994-1-102)

Composite dowels are an alternative shear connector to headed studs. In some countries experience with this type of shear connector exists but it was assumed to be not mature enough to be part of EN 1994-1-1.

So a Technical Specification was created taking into account different types of composite dowels as well as new research results.

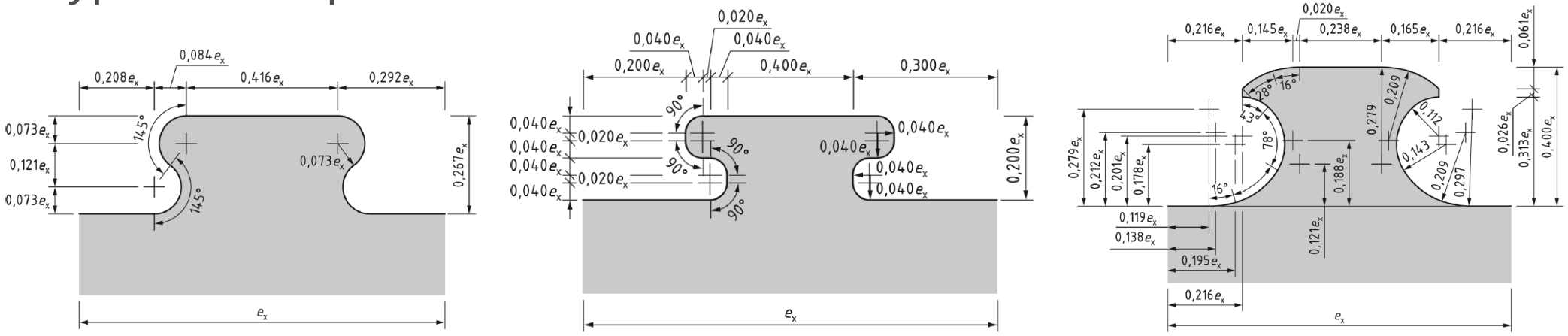



*Figure A.1 © BSI

# TS for composite dowels (CEN/TS 1994-1-102)

In the main part of the TS all known failure modes are covered in a general way. For each type of dowel an Annex provides specific information on failure modes and design equations.

## Annex A

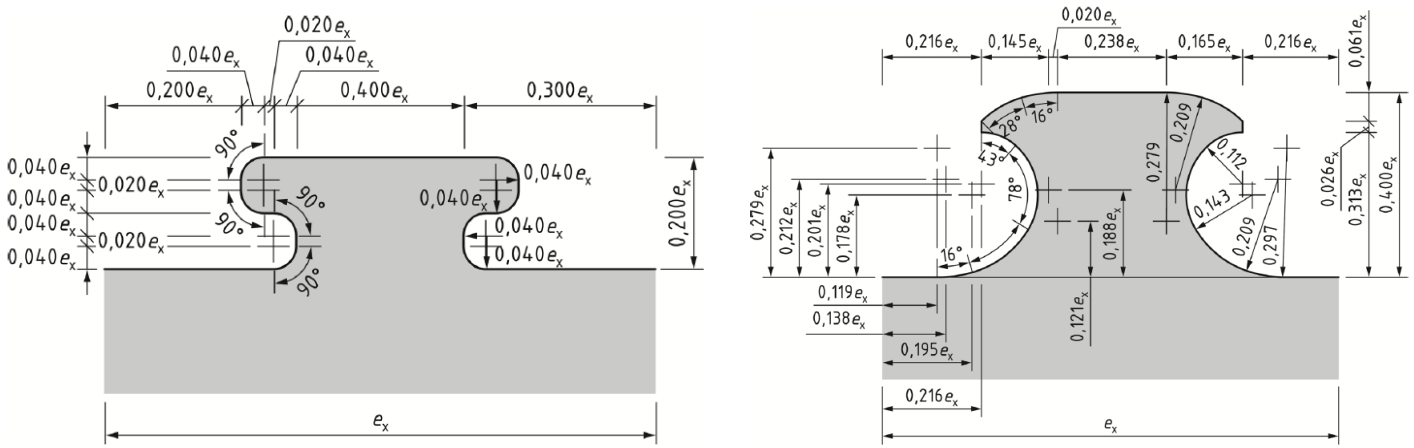



*Figure A.1 © BSI

## Annex B

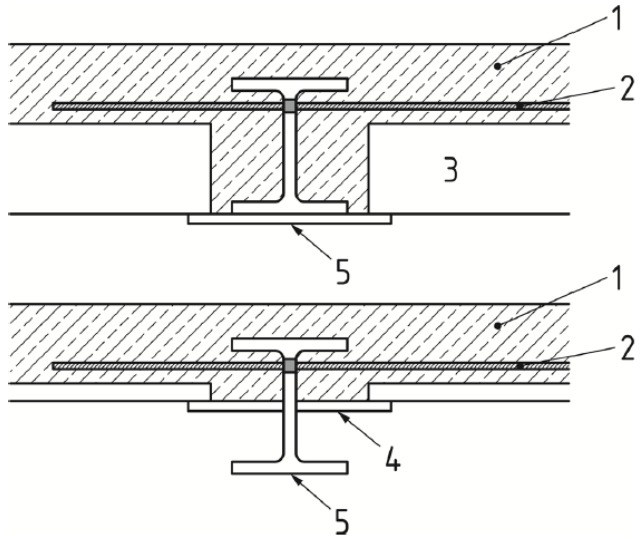
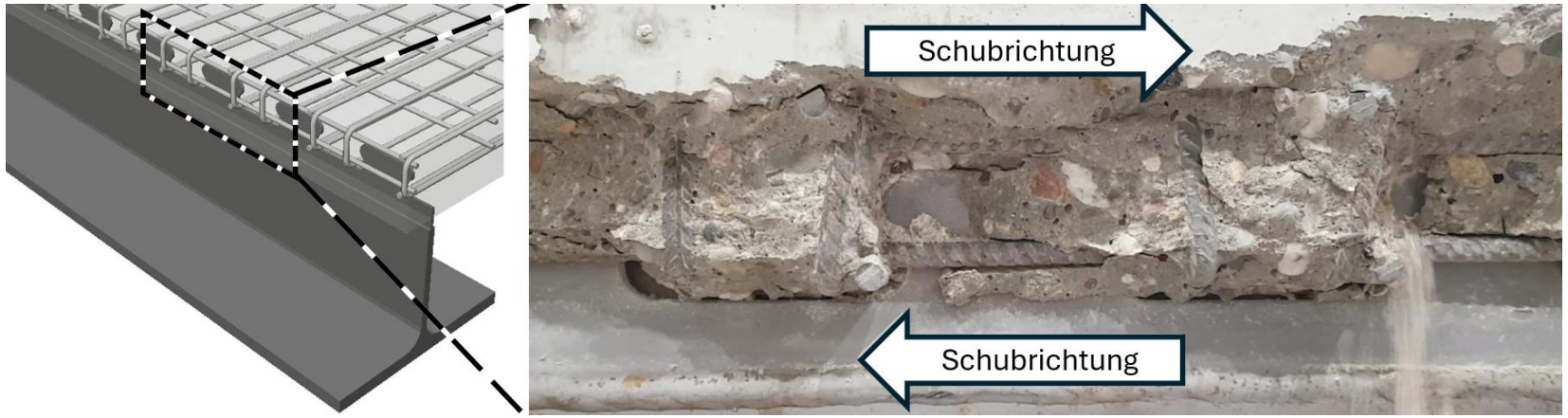



*Figure B.1 © BSI

# TS for composite dowels (CEN/TS 1994-1-102)

For puzzle shape dowel and clothoidal dowels a new failure mode was integrated, the so-called pry-out of a concrete edge $P_{poe,d}$.

This failure mode by splitting of thin concrete layers ( as shown here) or by spalling of the concrete cover at the edge of concrete webs.



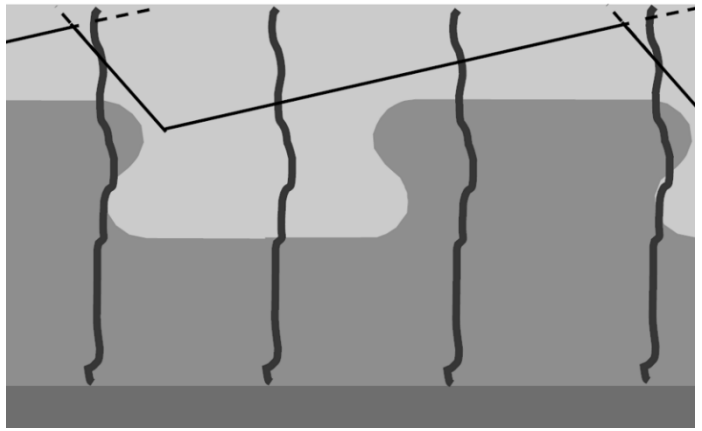

© Yannick Broschart

# TS for composite dowels (CEN/TS 1994-1-102)

Another import impact on the resistance to pry-out of a concrete cone is the influence of transverse cracks that divide the cone into several parts.

A reduction factor $\psi_{crack}$ was developed to take this effect into account. A specific challenge was the determination of crack width at ULS for calculation of vertical shear transfer through the cracks.

Simplified rules are provided for ease of use in design practice.



© Wolfgang Kurz, Manuel Risch

# TS for composite dowels (CEN/TS 1994-1-102)

For fatigue design rules for specific dowel geometries were developed and also evaluated for weathering steel.

Also rules for welds and splices of those composite dowels were integrated.

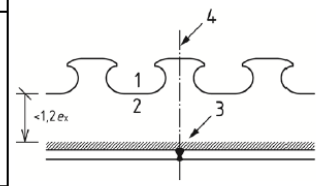
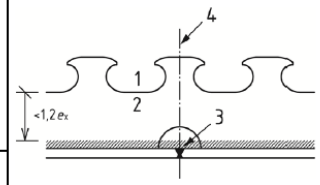
| Detail category | Constructional detail | | Symbol | Description | Supplementary Requirements |
|---|---|---|---|---|---|
| 112 |  | 7 |  | **7** Flange and web splices in plate girders, welded from both sides, ground flush | **7, 8, 9:** No full cross-section joint. Splices welded before assembly of girder. Longitudinal weld should be checked using Table 10.3 **7**: see **1**. **8**: see **2**. **9**: see **3**. |
| 90 |  | 8 |  | **8** as aforementioned, but as-welded, With flank angle ≥ 150° | |
| 80 | | 9 |  | **9** as aforementioned, but as-welded, with flank angle ≥ 110° | |
| 90 |  | 10 |  | **10** Full cross-section but welds of rolled sections of same dimensions with out tolerance difference, ground flush, with semi-circular cope holes | Rolled sections cut and rewelded. See **1**. |
| 80 | | 11 |  | **11** as aforementioned, but as-welded, with flank angle ≥ 150° or with tolerance differences | See **2**. |

*Table A.1© BSI

# TS for composite dowels (CEN/TS 1994-1-102)

For the dowels in Annex B the TS provides advanced calculation rules to EN 1994-1-1.

A main issue was the adaption of the results of several test series to the new ductility requirements and definitions given in EN 1994-1-1.

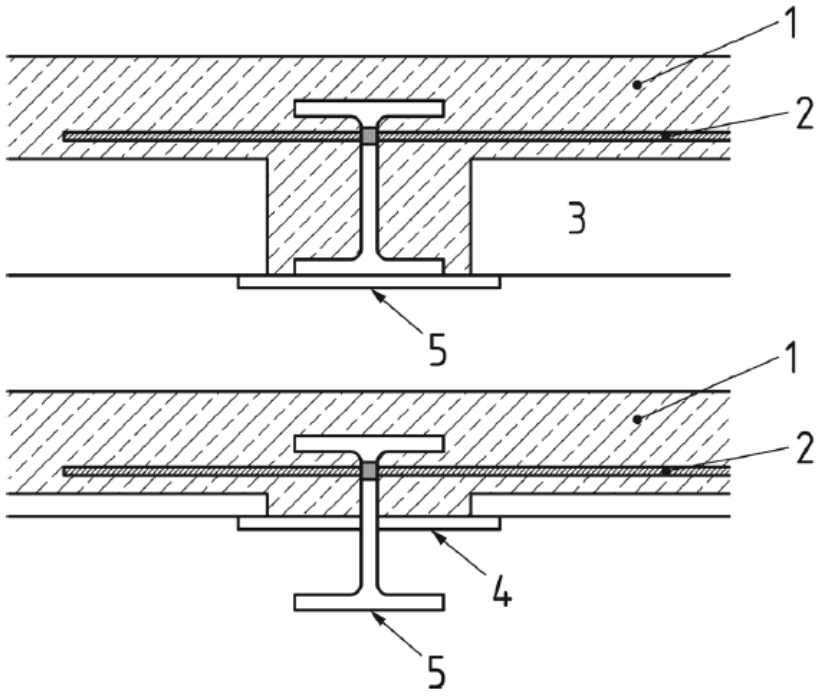



*Figure B.1 © BSI

# EN 1994 ‘Eurocode 4: Design of composite steel and concrete structures’

Stephen Hicks, Mike Banfi, Wolfgang Kurz, François Hanus; CEN/TC 250/SC 4

EUROCODES

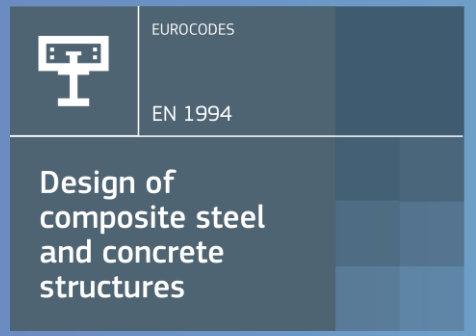
EUROCODES

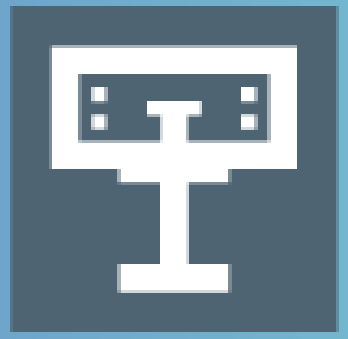
EN 1994

Design of composite steel and concrete structures

# Part 1-2

Stephen Hicks, Mike Banfi, Wolfgang Kurz, François Hanus

# Sub-Chapters and Annexes

*Existing Annexes → Normative*

*Existing Annexes → Informative*

*New Annexes → Normative*

1. Scope
2. Normative References
3. Terms, definition and sym[illegible]s
4. Basis of design
5. Material pr[illegible]ties
6. Tabu[illegible]d design data
7. [illegible]mplified design data
8. Advanced design methods
9. Detailing

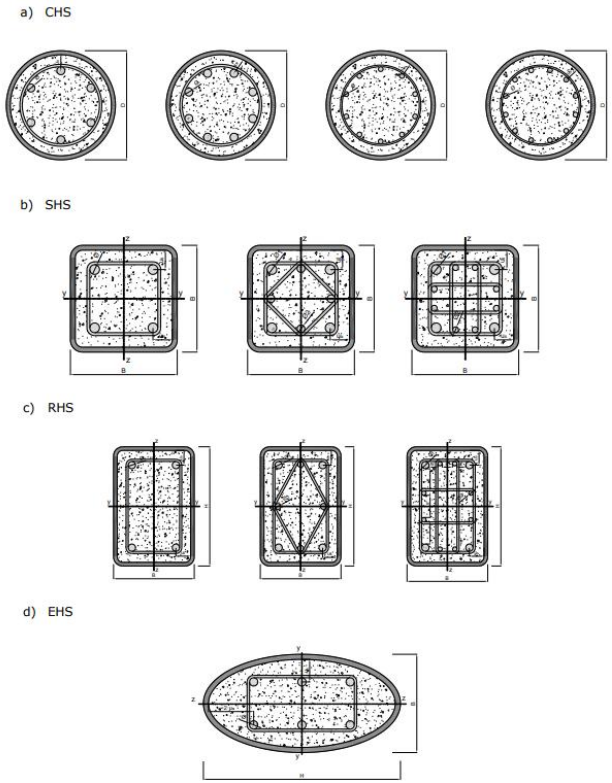
Harmonization of sub-Chapters

A. Strain-hardening of structural steel
B. Unprotected slabs
C. Composite beams (w/o encasement)
D. Composite beams (with encasement)
E. Partially-encased columns
Concrete filled hollow sections
G. Tensile Membrane Action
H. Shallow floor beams
I. Beams with large web openings

# Annex F : Concrete-filled tubular sections

Simplified design methods currently → New method with extended scope

1. limited to circular and square sections
2. limited to columns with relative slenderness lower than or equal to 0,5 (corrig. A1:2014)
3. limited to concrete classes up to C40/50




Source : Final Report FRISCC, RFCS Project 2012-2015

# Annex F : Concrete-filled tubular sections

Thermal analysis based on simplified distribution of T°

1. $\theta_{a,eq}$ for tubular section
2. $\theta_{c,eq}$ for concrete core
3. $\theta_{s,eq}$ for reinforcing bars

Concrete core:

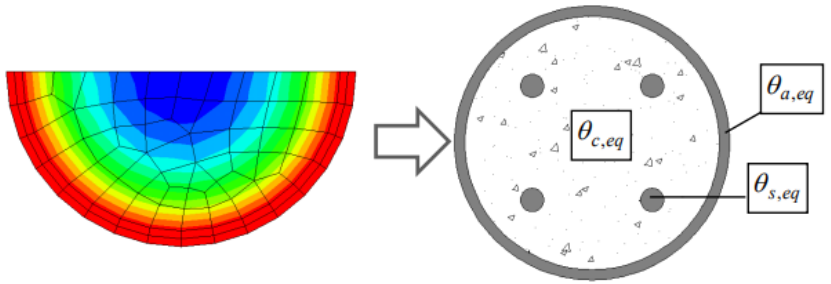
$$\theta_{c,eq} = 81{,}8 - 5{,}05 \cdot t_{fi} + 0{,}003 \cdot {t_{fi}}^2 - \frac{15{,}07 A_m}{V} + 0{,}3 \left(\frac{A_m}{V}\right)^2 - 0{,}88 \cdot t_{fi} \cdot A_m/V + 7{,}43 \cdot {t_{fi}}^{0{,}842} \cdot (A_m/V)^{0{,}714} \quad \text{(F.1)}$$

Steel section:

$$\theta_{a,eq} = -824{,}67 - 5{,}58 \cdot t_{fi} + 0{,}007 \cdot {t_{fi}}^2 - 0{,}01 \cdot t_{fi} \cdot A_m/V + 645{,}08 \cdot {t_{fi}}^{0{,}269} \cdot (A_m/V)^{0{,}017} \quad \text{(F.2)}$$

Reinforcing bars:

$$\theta_{s,eq} = \beta_3 \cdot \left(\frac{t_{fi}}{u_s^2}\right)^3 + \beta_2 \cdot \left(\frac{t_{fi}}{u_s^2}\right)^2 + \beta_1 \cdot \left(\frac{t_{fi}}{u_s^2}\right) + \beta_0 \quad \text{(F.3)}$$




Real temperature field — Equivalent temperature field

Source : Final Report FRISCC, RFCS Project 2012-2015

## Scope

$5 \le A_m/V \le 30$

$10 \le D/t_w \le 60$

$5 \le l_{fi}/D \le 30$

| | | $u_s$ [mm] | $\beta_3$ | $\beta_2$ | $\beta_1$ | $\beta_0$ |
|---|---|---|---|---|---|---|
| CHS | | *20* | 7236,5 | -10458 | 5497,6 | 19,38 |
| | | *30* | 58714 | -41328 | 10910 | 11,179 |
| | | *35* | 0 | -12732 | 6518 | 91,208 |
| | | *50* | 0 | -55639 | 13768 | -19,897 |
| | | *55* | 0 | -43201 | 10790 | 24,229 |
| | | *70* | 0 | 0 | 8858 | 96,676 |

# Annex F : Concrete-filled tubular sections

Axial buckling load based on existing simplified method

1. Plastic resistance to axial compression $N_{pl,Rd,fi}$
2. Effective flexural stiffness $(EI)_{eff,fi}$ accounting for flexural stiffness correction factors $\varphi_{i,\theta}$
3. Buckling curve a (unreinforced) or b (reinforced) of EN 1993-1-1

$$N_{pl,Rd,fi} = A_a f_{y,\theta}(\theta_{a,eq})/\gamma_{M,a,fi} + A_c f_{c,\theta}(\theta_{c,eq})/\gamma_{M,c,fi} + A_s f_{s,\theta}(\theta_{s,eq})/\gamma_{M,s,fi} \quad \text{(F.4)}$$

$$(EI)_{eff,fi} = \varphi_{a,\theta} E_{a,\theta}(\theta_{a,eq}) I_a + \varphi_{c,\theta} E_{c,\theta}(\theta_{c,eq}) I_c + \varphi_{s,\theta} E_{s,\theta}(\theta_{s,eq}) I_s \quad \text{(F.5)}$$

Concrete core: $\varphi_{c,\theta} = 1.2$

| CHS |
| --- |
| $\varphi_{a,\theta} = 0{,}75 - 0{,}023 \cdot (A_m/V)$ |

| CHS & SHS |
| --- |
| $\varphi_{s,\theta} = 0{,}8 - 0{,}002 \cdot t_{fi}$ |

$$N_{cr,fi} = \pi^2 (EI)_{eff,fi} / l_{fi}^2 \quad \text{(F.6)}$$

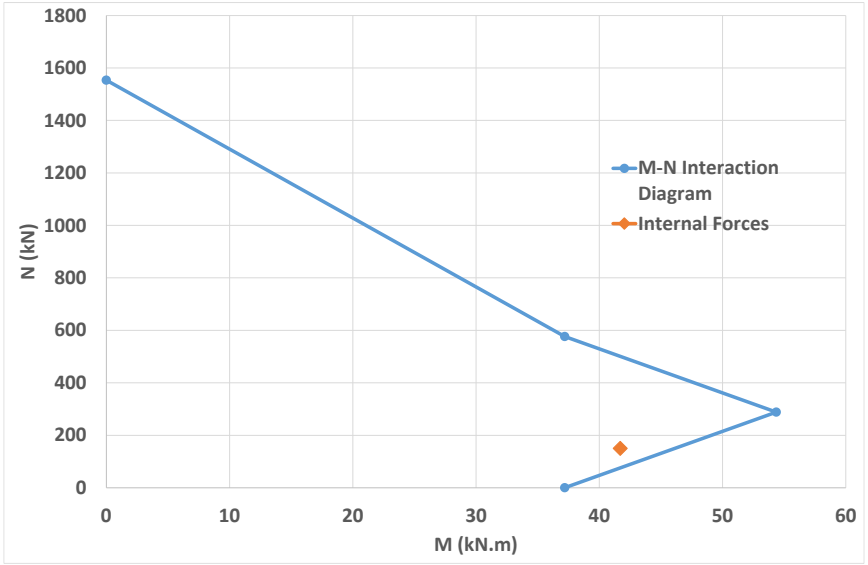
$$\bar{\lambda}_\theta = \sqrt{N_{pl,R,fi} / N_{cr,fi}} \quad \text{(F.7)}$$

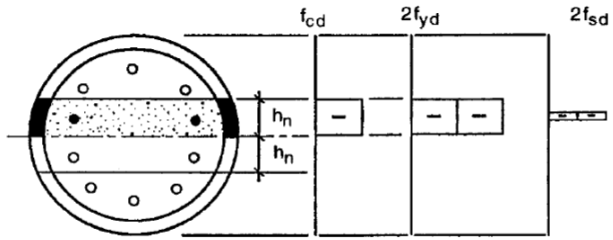
$$N_{Rd,fi} = \chi N_{pl,Rd,fi} \quad \text{(F.8)}$$

# Annex F : Concrete-filled tubular sections

Design buckling load in the fire situation in case of eccentric loading

1. Construction of M-N interaction diagram at elevated temperature
2. Euler buckling load, based on effective flexural stiffness $(EI)_{eff,II,fi}$ for second-order effects
3. Verification of applied internal forces ($N_{Ed,fi}$ ; $M_{Ed,fi}$) by iterations




$$h_n = \frac{A_c \cdot f_c - A_{sn} \cdot (2f_s - f_c)}{2D \cdot f_c + 4t \cdot (2f_y - f_c)}$$

Source : Bergmann R et al. Design guide for CFHS columns under static and seismic loading, CIDECT, 1995.

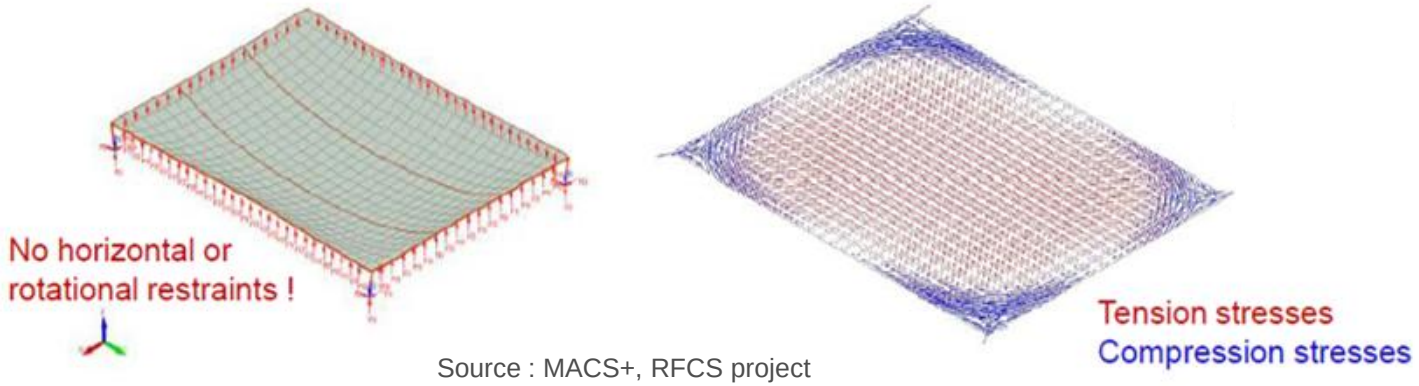
$$(EI)_{eff,II,fi} = K_\theta \cdot K_o \cdot \left[ \varphi_{a,\theta} \cdot E_{a,\theta}\left(\theta_{a,eq}\right) I_a + \varphi_{s,\theta} \cdot E_{s,\theta}\left(\theta_{s,eq}\right) I_s + K_{e,II} \cdot \varphi_{c,\theta} \cdot E_{c,\theta}\left(\theta_{c,eq}\right) I_c \right] \quad \text{(F.9)}$$

# Annex G : Tensile Membrane Action

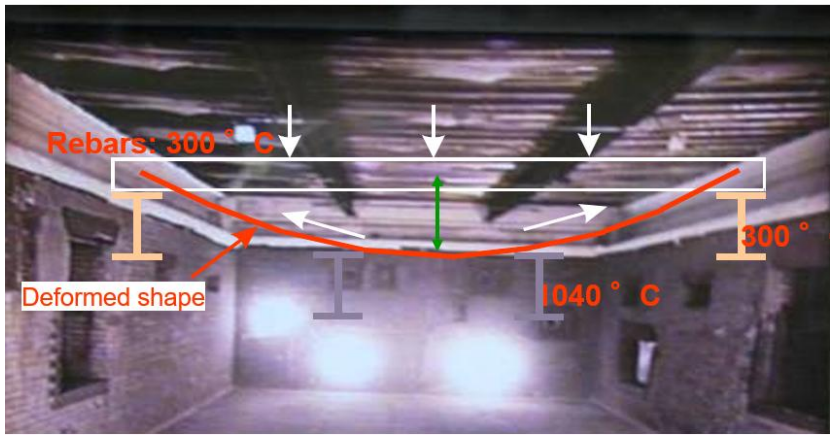
Basic principles

1. Tensile membrane (rebars)
2. Compression ring (concrete slab)
3. Perimeter beams or supports

Source : BRE Cardington Fire Test Report




Source : MACS+, RFCS project

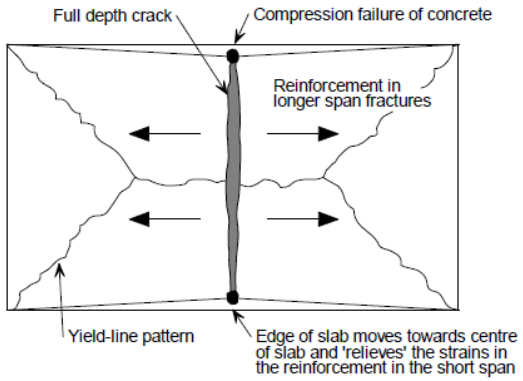


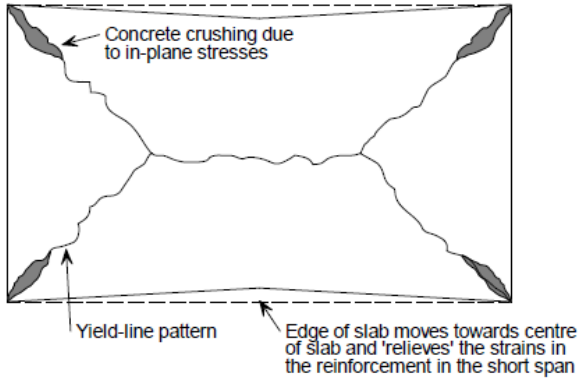

Source : FRACOF Fire Test (Efectis)

# Annex G : Tensile Membrane Action

Failure Modes

1. Tensile failure of mesh reinforcement
2. Compressive failure of concrete
3. Bearing capacity of perimeter beams or supports



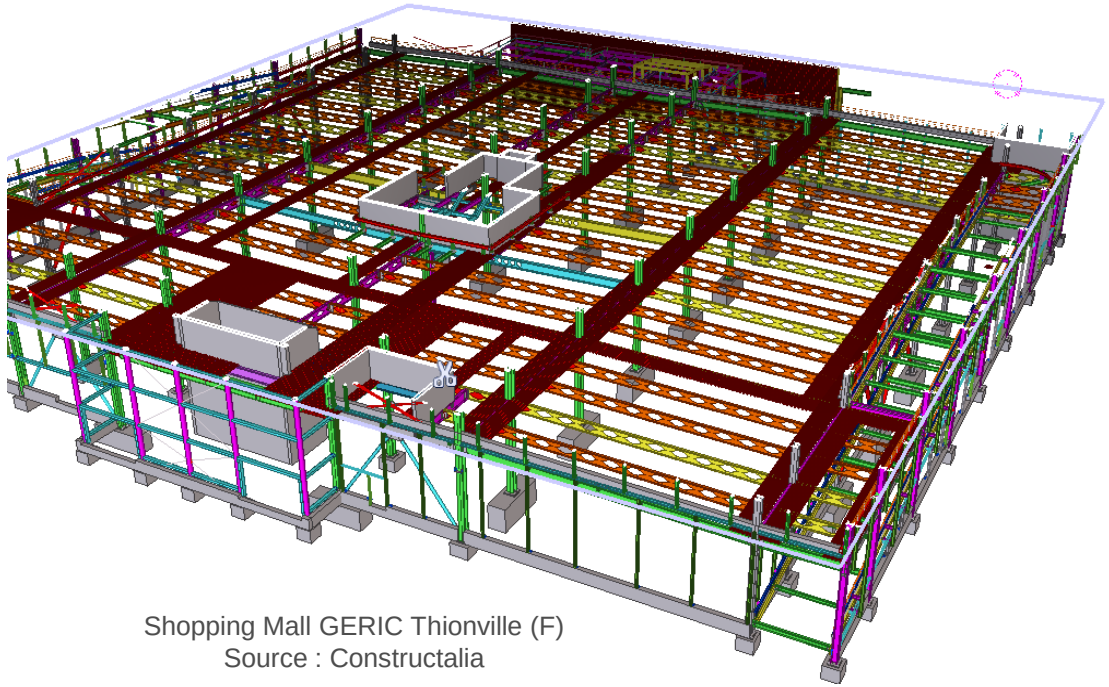




Source : MACS+, RFCS project

# Annex G : Tensile Membrane Action

Ground level (shopping)

→ partial fire protection



Shopping Mall GERIC Thionville (F)
Source : Constructalia

Above levels (parking)

→ no fire protection

# Annex G : Tensile Membrane Action

Step-by-step procedure

1. Distribution of temperature

covered by EN 1993-1-2 and EN 1994-1-2

# Annex G : Tensile Membrane Action

Step-by-step procedure

1. Distribution of temperature
2. Calculation of slab maximum deflection

$$w = min\left\{\frac{\alpha_c(\theta_2-\theta_1)\ell^2}{19.2h_{eff}} + min\left[\sqrt{\left(\frac{0.5f_{sy}}{E_s\gamma_{M,s,fi}}\right)\frac{3L^2}{8}};\frac{\ell}{30}\right];\frac{L+\ell}{30}\right\}$$

Thermal bowing of slab due to temperature gradient

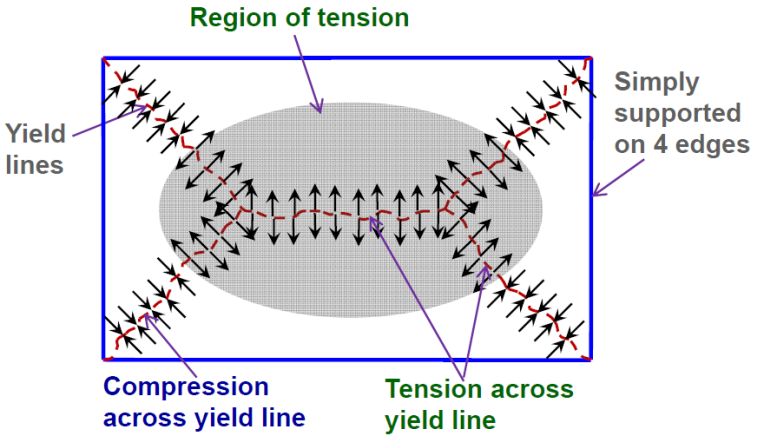
Deformation of the slab under transversal loads

Maximal deflection of the system

# Annex G : Tensile Membrane Action

Step-by-step procedure

1. Distribution of temperature
2. Calculation of slab maximum deflection
3. Calculation of composite slab loadbearing capacity



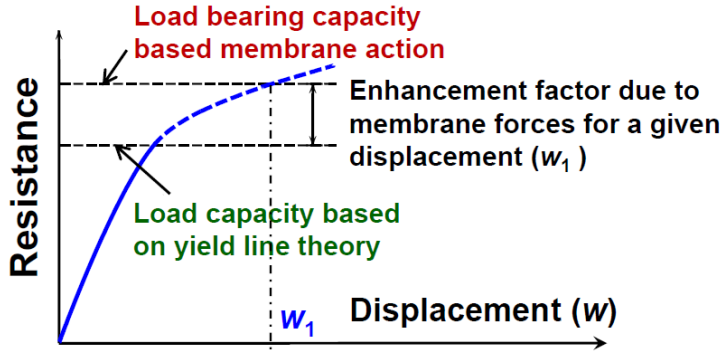

$$q_{\mathrm{Rd,fi}} = \frac{6M_{\mathrm{o,fi}}}{n^2 L^2}$$

Reference load bearing capacity of the slab

$$q_{Rd,slab,fi} = e \times q_{Rd,fi}$$

$$e = (e_{1b} + e_{1m}) - \frac{(e_{1b} + e_{1m}) - (e_{2b} + e_{2m})}{1 + 2\mu a^2}$$

Overall enhancement factor due to activation of TMA




Source : MACS+, RFCS project

# Annex G : Tensile Membrane Action

Step-by-step procedure

1. Distribution of temperature (covered by EN 1993-1-2 and EN 1994-1-2)
2. Calculation of slab maximum deflection
3. Calculation of composite slab loadbearing capacity
4. Calculation of loadbearing capacity of unprotected secondary beams

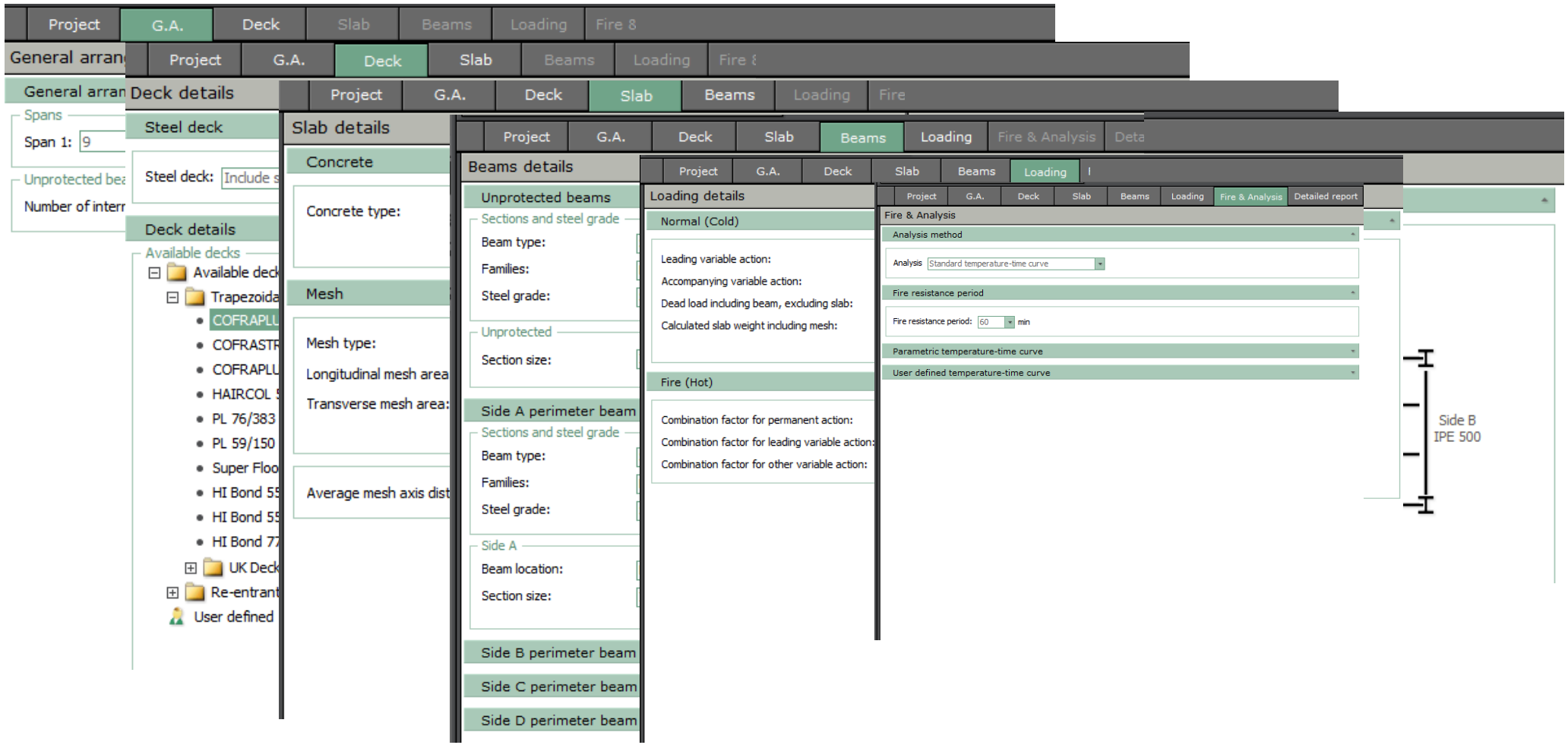
$$q_{\mathrm{Rd,ub,fi}} = \frac{8}{{L_1}^2} \min_{j=1,n_{\mathrm{ub}}} \left( \frac{M_{\mathrm{Rd},j,fi}}{b_j} \right)$$

# Annex G : Tensile Membrane Action

Step-by-step procedure

1. Distribution of temperature (covered by EN 1993-1-2 and EN 1994-1-2)
2. Calculation of slab maximum deflection
3. Calculation of composite slab loadbearing capacity
4. Calculation of loadbearing capacity of unprotected secondary beams
5. Verification of the resistance of perimeter beams

# Annex G : Tensile Membrane Action



# Annex G : Tensile Membrane Action

**8. Summary output**

- Default mesh direction

| | | |
|---|---|---|
| Maximum unity factor: | 0.94 | **Floor slab a** |
| Factored load in fire: | 5.98 kN/m² | |
| Fire curve: | Standard temperatur | |

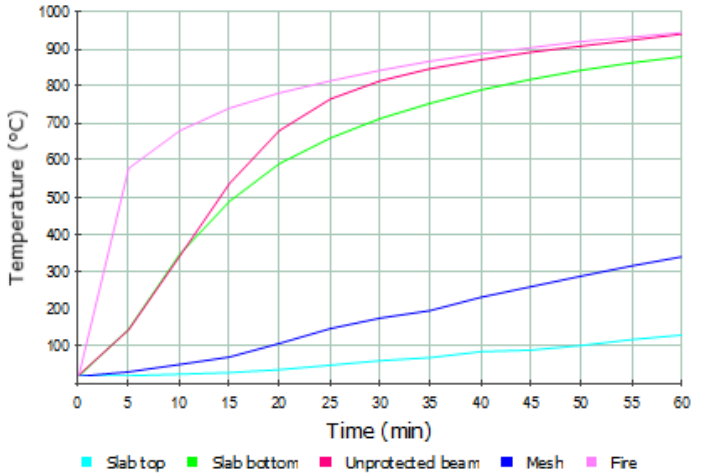
**9. Default mesh direction**

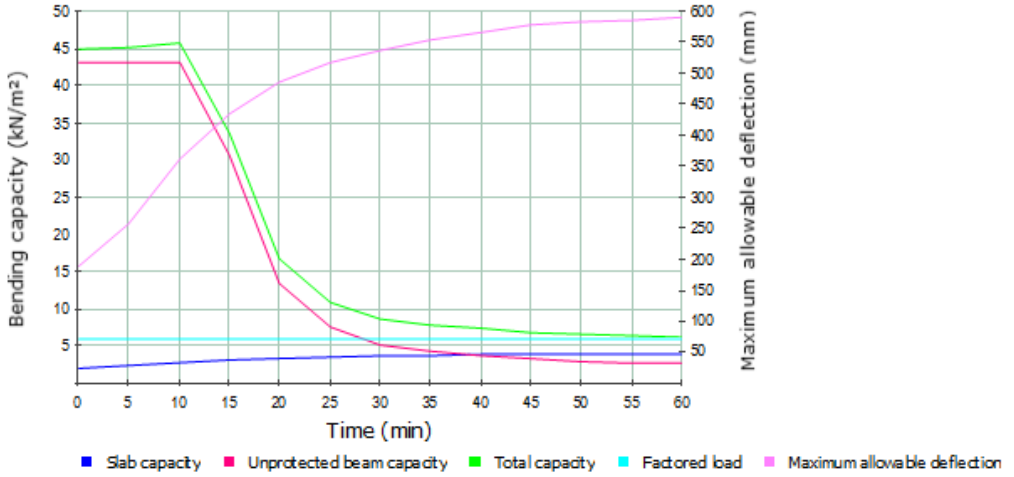
| | | |
|---|---|---|
| Longitudinal mesh area: | 142 mm²/m | Ba |
| Transverse mesh area: | 142 mm²/m | Ba |
| Factored load in fire: | 5.98 kN/m² | |

- Tabular results

| Time | Beam | Mesh | Slab top | Slab bottom | Beam capacity | Max allow defl |
|---|---|---|---|---|---|---|
| mins | °C | °C | °C | °C | kN/m² | m |
| 0 | 20 | 20 | 20 | 20 | 43.27 | 1 |
| 5 | 141 | 31 | 20 | 147 | 43.27 | 2 |
| 10 | 341 | 53 | 22 | 348 | 43.27 | 3 |
| 15 | 536 | 75 | 28 | 491 | 30.74 | 4 |
| 20 | 679 | 109 | 36 | 592 | 13.46 | 4 |
| 25 | 765 | 149 | 48 | 663 | 7.44 | 5 |
| 30 | 814 | 180 | 62 | 716 | 5.05 | 5 |
| 35 | 846 | 200 | 71 | 758 | 4.27 | 5 |
| 40 | 871 | 236 | 83 | 791 | 3.67 | 5 |
| 45 | 891 | 266 | 89 | 820 | 3.18 | 5 |
| 50 | 909 | 293 | 103 | 844 | 2.87 | 5 |
| 55 | 924 | 319 | 119 | 865 | 2.72 | 5 |
| 60 | 938 | 345 | 131 | 884 | 2.58 | 5 |

**10. Graphical output**




**adequate**

| | | | |
|---|---|---|---|
| | Solid beam | Non composite | Edge beam |
| | IPE 500 | | |
| re situation: | 252.63 kNm | | |
| | 24.95 kN/m | | |
| | 0.34 | | |
| | 655 °C | | |
| | Solid beam | Composite | Internal beam |
| | IPE 500 | | |
| re situation: | 349.37 kNm | | |
| | 34.51 kN/m | | |
| | 80 % | | |
| | 0.27 | | |
| | 697 °C | | |
| | Solid beam | Composite | Internal beam |
| | IPE 500 | | |
| re situation: | 252.63 kNm | | |
| | 24.95 kN/m | | |
| | 80 % | | |
| | 0.2 | | |
| | 750 °C | | |
| | Solid beam | Non composite | Edge beam |
| | IPE 500 | | |
| re situation: | 349.37 kNm | | |
| | 34.51 kN/m | | |
| | 0.47 | | |
| | 602 °C | | |

# Annex H : Shallow Floor Beams

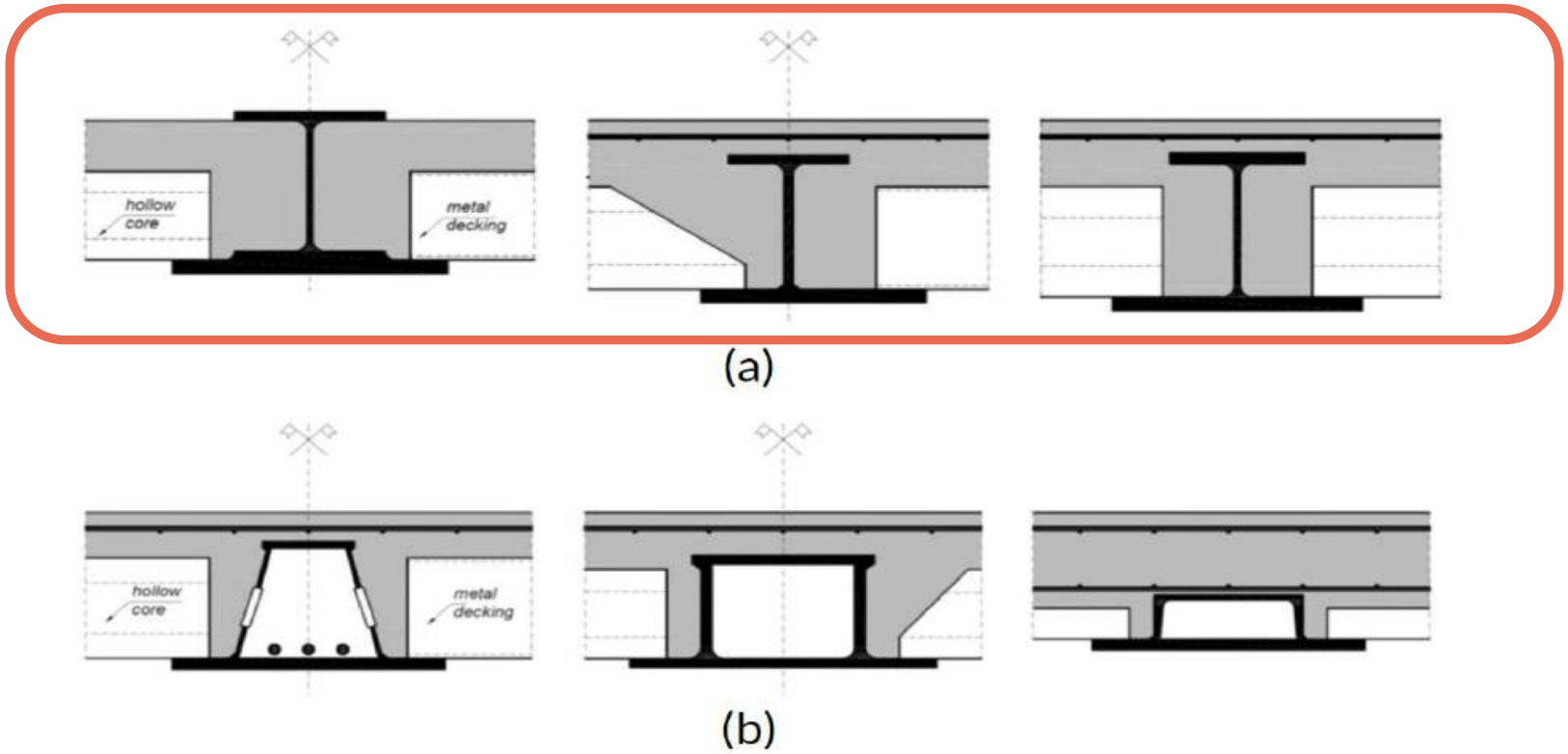



Figure 1 Examples of shallow floor cross-sections (a) open cross-sections (b) closed cross-sections [51].

Source : S. Hicks et al., New Eurocode 4 Design Rules for Shallow Floor Construction, 9th European Conference on Steel and Composite Structures – Eurosteel. Sheffield, UK, (2021)

# Annex H : Shallow Floor Beams

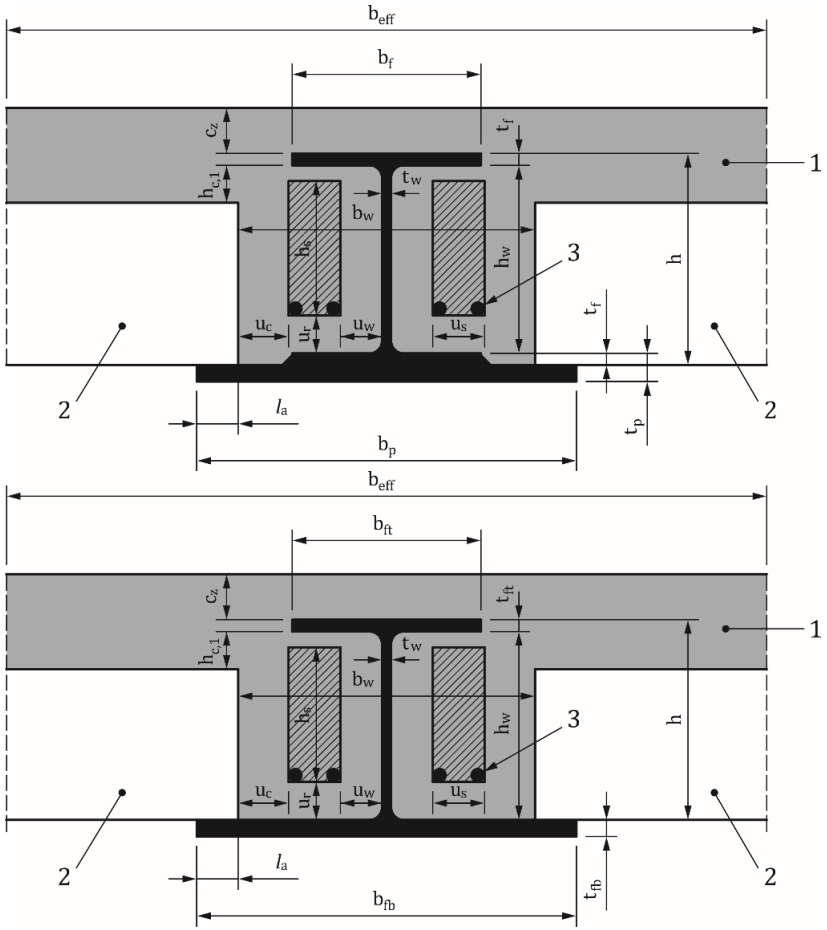



*Figure H.1 a & b © BSI

**Table H.1 — Field of application**

| Steel section | Concrete slab | Reinforcement |
|---|---|---|
| $10\ mm \le t_p \le 40\ mm$ (Type A)<br>$10\ mm \le t_{ft} \le 40\ mm$ (Type B) | $30\ mm \le c_z \le 150\ mm$ | $u_r = 30\ mm$ |
| $8\ mm \le t_f \le 40\ mm$ (Type A)<br>$8\ mm \le t_{fb} \le 40\ mm$ (Type B) | $b_f + 60\ mm \le b_w$ (Type A)[a]<br>$b_{ft} + 60\ mm \le b_w$ Type B)[a] | $u_c = 40\ mm$ |
| $0.7 \le t_p / t_f \le 2$ (Type A) | $20\ mm \le h_{c,1}$ | $u_w = 40\ mm$ |
| $6\ mm \le t_w \le 30\ mm$ | $40\ mm \le l_a$ | $h_s \le h_w - 2 \cdot u_r$ |
| $160\ mm \le h \le 450\ mm$<br>$160\ mm \le b_f \le 450\ mm$ (Type A)<br>$160\ mm \le b_{ft} \le 450\ mm$ (Type B) | | $A_s \le \begin{cases} 0{,}5 \cdot b_p \cdot t_p (TypeA) \\ 0{,}5 \cdot b_{fb} \cdot t_{fb} (TypeB) \\ 0{,}05 \cdot b_w \cdot (h + c_z) \\ 2 \cdot b_{eff} \cdot c_z \cdot \frac{f_{ck}}{f_{sk}} \end{cases}$ |
| $160\ mm \le b_p - b_f \le 250\ mm$ (Type A)<br>$110\ mm \le b_{fb} - b_{ft} \le 250\ mm$ (Type B) | | |

[a] The value of 60 mm may be reduced to 30 mm in case of a solid slab.

# Annex H : Shallow Floor Beams

Step-by-step procedure

1. Calculation of distribution of temperature and reduced width of bottom plate/flange
2. Calculation of position of P.N.A.
3. Calculation of design sagging bending resistance $M_{Rd,fi,+}$

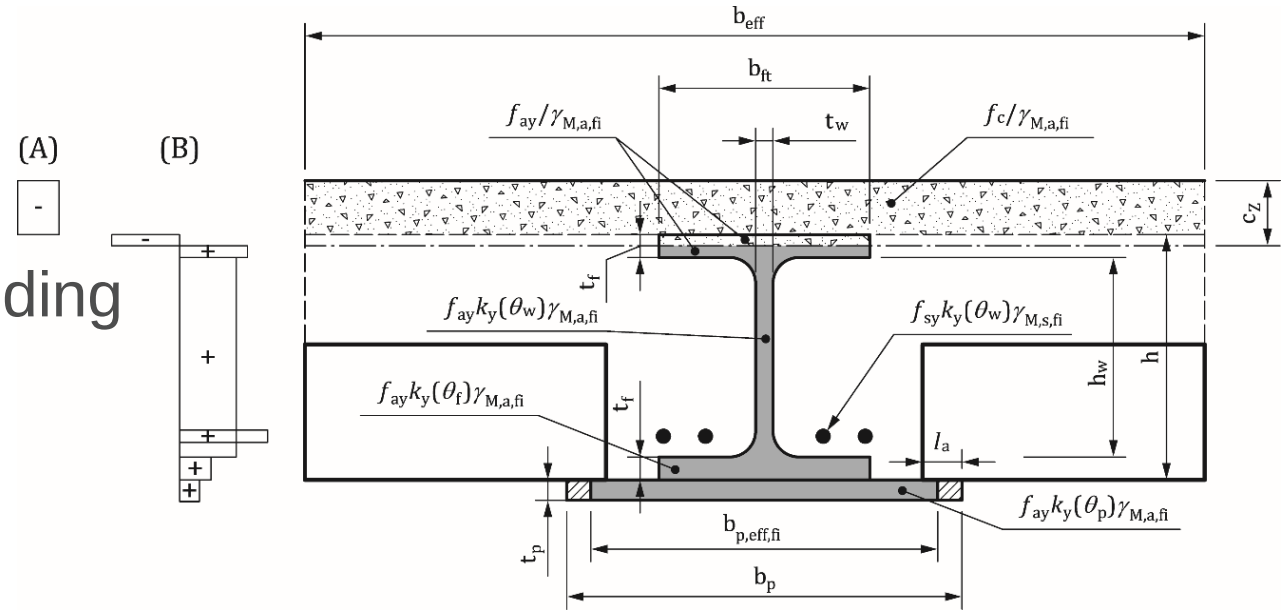



*Figure H.2 a & b © BSI

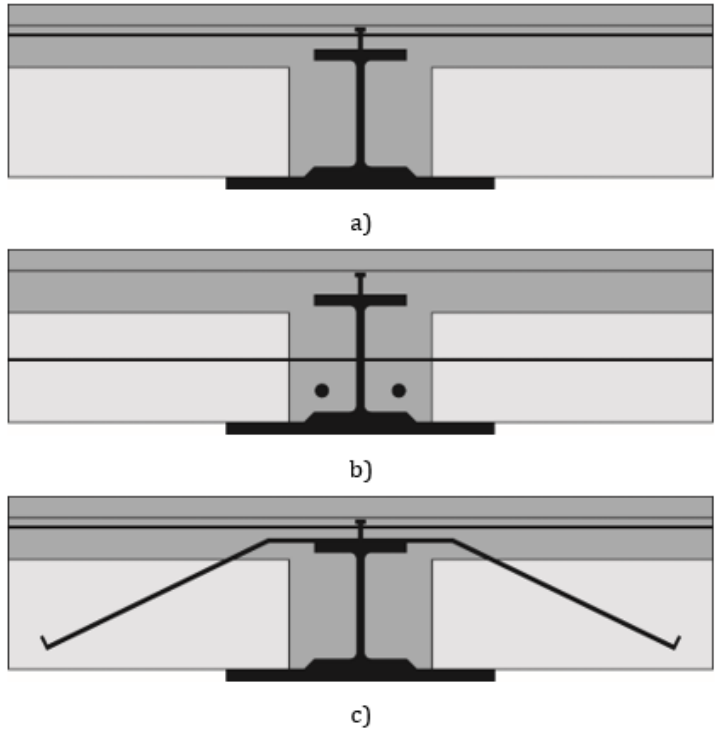
# Annex H : Shallow Floor Beams

Other considerations

1. Load transfer from concrete slab to the web of composite SFB. Potentially leading to reduced bearing capacity of bottom plate/flange

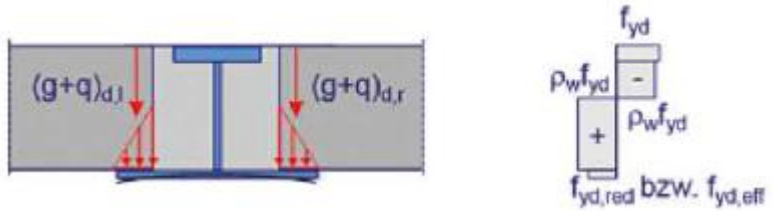
2. Protected beams covered by definition of specific section factors $A_{p,i} / V_i$ for bottom plate/flange

3. Annex may apply to non-composite SFB




a) direct bearing by the bottom flange or plate
b) additional reinforcement through the web
c) additional reinforcement above the top flange

*Figure 9.3 © BSI




Source : M. Schäfer, Zur Biegebemessung von Flachdecken in Verbundbauweise – Ergänzende Bemessungsregeln für Slim-Floor Träger, Stahlbau 84 (2015), Heft 4

# Annex I : Beams with large web openings

## Thermal analysis of steel beam with LWO (according to part 1-2 of EC3)

## Option 1 : Uniform temperature per failure mode

- Flexural bending in a gross section (no opening): from case (1) in Table E.1
- Flexural bending in a net section (with an opening) from case (2) in Table E.1
- *Vierendeel* bending from case (3) in Table E.1 for the top chord or (4) for the bottom chord
- Web post buckling: from case (5) in Table E.1 for the top chord or (6) for the bottom chord
- Web horizontal shear or weld shear at the junction of the chords: from case (7) in Table E.1
- Web vertical shear: from case (8) in Table E.1.

| | | |
|---|---|---|
| *Vierendeel* bending in the top chord (3) | 3 sides | $(A_m/V)_{3,3sides} = \dfrac{b_{top} + 2\left[\frac{a_0}{2}\left(\frac{1}{\cos\phi_{top}} - 1\right) + \frac{h_{w,top}}{\cos\phi_{top}} + t_{f,top}\right]}{b_{top}t_{f,top} + \left[\frac{a_0}{2}\left(\frac{1}{\cos\phi_{top}} - 1\right) + \frac{h_{w,top}}{\cos\phi_{top}}\right]t_{w,top}}$ |
| | 4 sides | $(A_m/V)_{3,4sides} = \dfrac{2\left[b_{top} + \frac{a_0}{2}\left(\frac{1}{\cos\phi_{top}} - 1\right) + \frac{h_{w,top}}{\cos\phi_{top}} + t_{f,top}\right]}{b_{top}t_{f,top} + \left[\frac{a_0}{2}\left(\frac{1}{\cos\phi_{top}} - 1\right) + \frac{h_{w,top}}{\cos\phi_{top}}\right]t_{w,top}}$ |

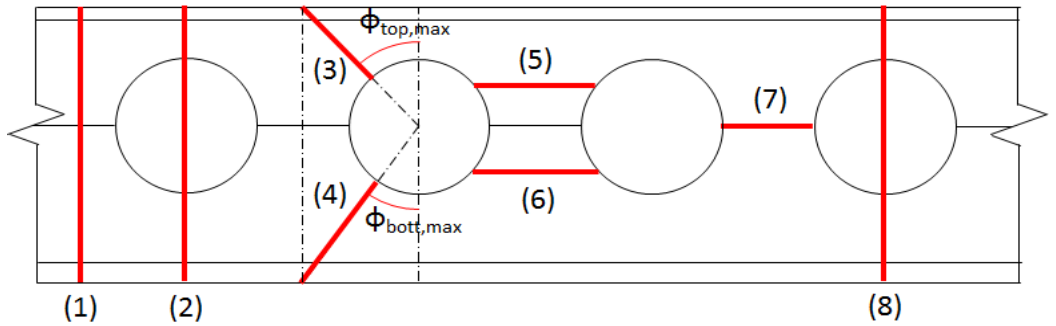



*Figure E.1 © DIN

$$\Delta\theta_{a,t} = k_{sh}\frac{A_m/V}{c_a\rho_a}\dot{h}_{net,d}\Delta t$$

Extracts from FprEN 1993-1-2:2023

# Annex I : Beams with large web openings

Thermal analysis of steel beam with LWO (according to part 1-2 of EC3)

Option 2 : Non-uniform distribution of temperature

| Element | Fire exposure | Section Factor, A/V |
|---|---|---|
| Top flange | 3 sides | $\frac{b_{top} + 2t_{f,top}}{b_{top}t_{f,top}}$ |
| | 4 sides | $2\frac{b_{top} + t_{f,top}}{b_{top}t_{f,top}}$ |
| Bottom flange | 4 sides | $2\frac{b_{bott} + t_{f,bott}}{b_{bott}t_{f,bott}}$ |
| Web | - | $\frac{4}{t_{w,top} + t_{w,bott}}$ |

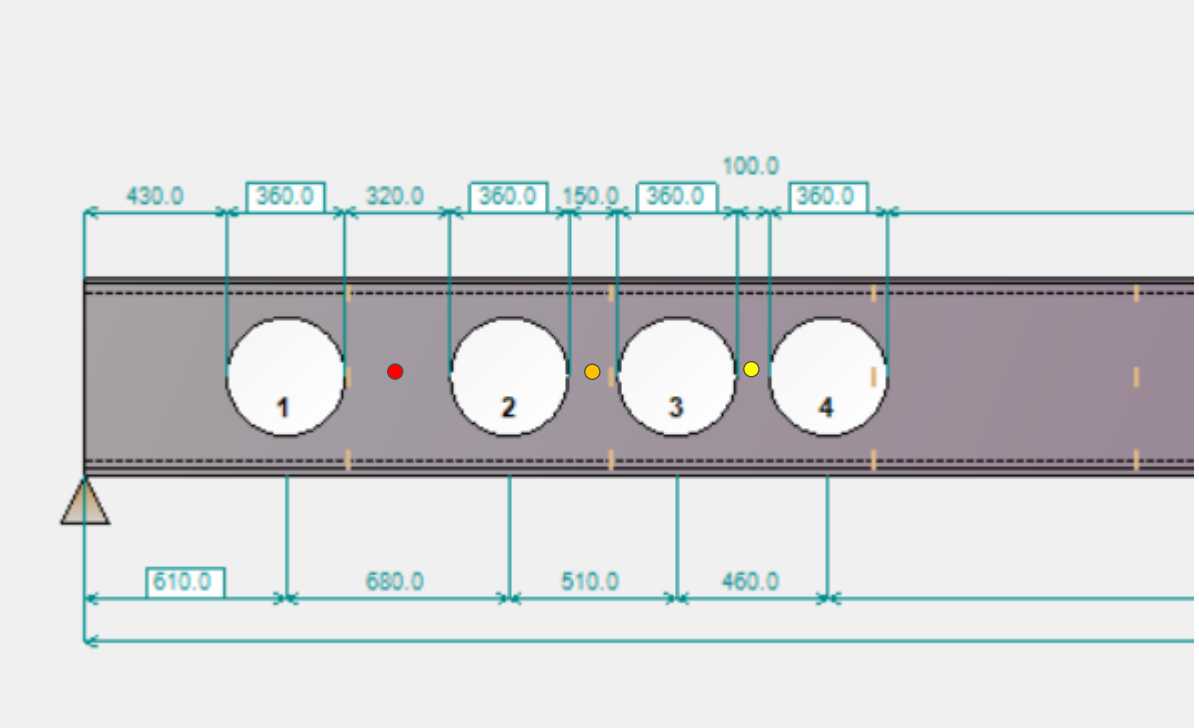


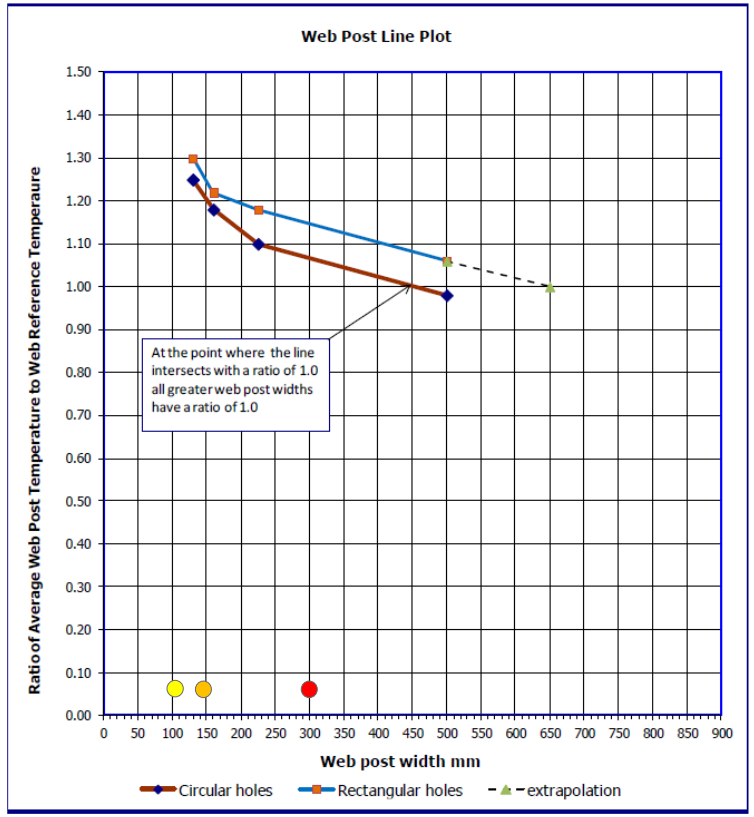
Extracts from FprEN 1993-1-2:2023

# Annex I : Beams with large web openings

Thermal analysis of steel beam with LWO (according to part 1-2 of EC3)

Specific case for protected beams with use of reactive coatings



# Annex I : Beams with large web openings

Mechanical analysis

1. Configurations covered by Annex E (stiff slabs) of part 1-1 not covered by part 1-2
2. Steel failure modes covered by part 1-2 of Eurocode 3
3. Contribution of concrete slab according to part 1-2 of Eurocode 2
4. Effective widths for local bending and for shear slightly different at elevated T°

# Other novelties

1. Specific emissivity for hot-dip galvanised steel (Section 5)

**5.2.1.1 Emissivity coefficient**

(1) The values of surface emissivity $\varepsilon_m$ in relation to the different types of steel should be taken from Table 5.1.

**Table 5.1 — Surface emissivity of different types of steel**

| Type of steel | $\varepsilon_m$ ($\leq$ 500 °C) | $\varepsilon_m$ ($>$ 500 °C) |
|---|---|---|
| Carbon steel | 0,7 | |
| HDG steel[a] | 0,35 | 0,7 |

[a] Steel that has been hot-dip galvanized according to EN ISO 1461 and with steel composition according to Category A or B of EN ISO 14713-2:2020, Table 1.

# Other novelties

1. Specific emissivity for hot-dip galvanised steel (Section 5)
2. Harmonization of thermal and mechanical properties of concrete with Eurocode 2 (Section 5)

**5.2.2.2 Thermal conductivitv**

(1) The thermal c
(5.10):

$\lambda_c = 2 - 0{,}2451$

$\lambda_c = -0{,}02604$

$\lambda_c = 1{,}36 - 0{,}13$

where

$\theta_c$ is th

**5.2.2.4 Density**

(1) The vari
determined f

$\rho(\theta_c) = \rho$

$\rho(\theta_c) = \rho$

$\rho(\theta_c) = \rho$

$\rho(\theta_c) = \rho$

(2) The densi

$\rho_{c,LC} = 1$

**5.3.2.1 Thermal expansion**

(1) The thermal expansion Δl/l of cc
regardless of the concrete strength.
Siliceous aggregates:

$\Delta l / l = -1{,}8.10^{-4} + 9.10^{-6}$

$\Delta l / l = 14.10^{-3}$

Calcareous aggregates:

$\Delta l / l = -1{,}2.10^{-4} + 6.10^{-6}\,\theta_c +$

$\Delta l / l = 12.10^{-3}$

**Table 5.5 — Values for the two main parameters of the stress-strain relationships of normal weight concreteat elevated temperatures**

| Concrete temperature $\theta_c$ | $k_{c,\theta} = \frac{f_{c,\theta}}{f_{ck}}$ | $k_{c,\theta} = \frac{f_{c,\theta}}{f_{ck}}$ | $\varepsilon_{c1,\theta} \cdot 10^3$ | $\varepsilon_{cu1,\theta} \cdot 10^3$ |
|---|---|---|---|---|
| [°C] | Siliceous aggregates | Calcareous aggregates | | |
| 20 | 1 | 1 | 2,5 | 20,0 |
| 100 | 1 | 1 | 4,0 | 22,5 |
| 200 | 0,95 | 0,97 | 5,5 | 25,0 |
| 300 | 0,85 | 0,91 | 7,0 | 27,5 |
| 400 | 0,75 | 0,85 | 10,0 | 30,0 |
| 500 | 0,60 | 0,74 | 15,0 | 32,5 |
| 600 | 0,45 | 0,60 | 25,0 | 35,0 |
| 700 | 0,30 | 0,43 | 25,0 | 37,5 |
| 800 | 0,15 | 0,27 | 25,0 | 40,0 |
| 900 | 0,08 | 0,15 | 25,0 | 42,5 |
| 1 000 | 0,04 | 0,06 | 25,0 | 45,0 |
| 1 100 | 0,01 | 0,02 | 25,0 | 47,5 |
| 1 200 | 0 | 0 | — | — |

(5) The values of the the $\varepsilon_{c1,\theta}$ and $\varepsilon_{cu1,\theta}$ for lightweight aggregate concrete should be based on testing.

# Other novelties

1. Specific emissivity for hot-dip galvanised steel (Section 5)
2. Harmonization of thermal and mechanical properties of concrete with Eurocode 2 (Section 5)
3. Extension of Annex for composite slabs to decks with a top stiffener (Annex B)

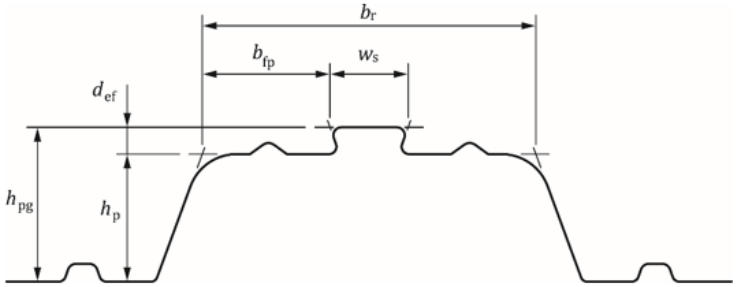



**Figure B.2 — Open trough sheeting with a top re-entrant stiffener**

(3) In case of open trough sheetings with a top re-entrant stiffener (Figure B.2), the fire resistance with respect to criterion "I" may be determined from Formulae (B.1) and (B.2), where $h_1$ and $h_2$ should respectively be replaced by $h_{1,mod}$ and $h_{2,mod}$, calculated according to Formulae (B.4) to (B.7).

$$h_{1,mod} = h_1 - d_{ef}/2 \quad \text{when} \quad \frac{w_s}{b_r} \le 0,35 \qquad \text{(B.4)}$$

$$h_{1,mod} = h_1 - d_{ef} \quad \text{when} \quad \frac{w_s}{b_r} > 0,35 \qquad \text{(B.5)}$$

$$h_{2,mod} = h_p + d_{ef}/2 \quad \text{when} \quad \frac{w_s}{b_r} \le 0,35 \qquad \text{(B.6)}$$

$$h_{2,mod} = h_p + d_{ef} \quad \text{when} \quad \frac{w_s}{b_r} > 0,35 \qquad \text{(B.7)}$$

# Thank you!