

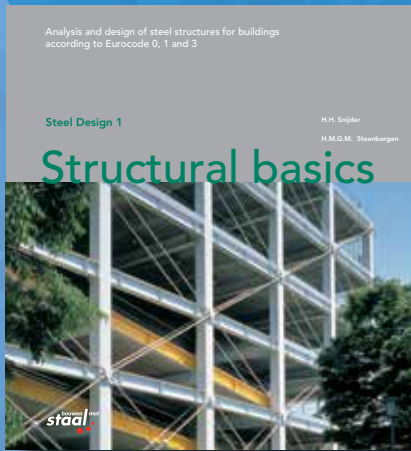
The Steel Design series is a common initiative of six European partners active in steel construction. It offers high-quality textbooks to develop an interest in steel structures and is intended for high school and university level students, as well as designers who are not yet specialised in steel and steel construction.

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- based on the EN version of the Eurocode
- easy to connect with the local National Annexes to the Eurocode





Structural basics

Steel Design 1

CONTENT

- 1 Structural safety
- 2 Actions and deformations
- 3 Modelling
- 4 Analysis
- 5 Analysis methods
- 6 Assessment by code checking
- 7 Resistance of cross-sections

H.H. Snijder and H.M.G.M. Steenbergen, *Structural basics. Analysis and design of steel structures for buildings according to Eurocode 0, 1 and 3 (Steel Design 1)*, Zoetermeer 2019, ISBN 978-90-72830-98-2, format 23x25 cm, 272 p.

Also available as ebook, ISBN 978-3-433-61069-5, at www.ernst-und-sohn.de/steel-design



Fire

Steel Design 2

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- 1 Fire safety
- 2 Calculation of the fire resistance
- 3 Fire safety engineering
- 4 Design tables

A.F. Hamerlinck, *Fire safety and fire resistant design of steel structures for buildings according to Eurocode 3 (Steel Design 2)*, Zoetermeer 2021, ISBN 978-90-75146-04-02, format 23x25 cm, 164 p.

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Connections

Steel Design 3

CONTENT

- 1 Connections in steel structures
- 2 Bolts in clearance holes
- 3 Slip-resistant connections, rivets and pins
- 4 Welds
- 5 Design and detailing of connections

J.W.B. Stark, *Connections. Behaviour of connections in steel structures and design of mechanical fasteners and welds according to Eurocode 3 (Steel Design 3)*, Zoetermeer 2021, ISBN 978-90-75146-05-0, format 23x25 cm, 206 p.

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Composite structures

Steel Design 4



CONTENT

- 1 Composite structures
- 2 Composite beams
- 3 Composite slabs
- 4 Composite columns
- 5 Composite joints
- 6 Worked example cinema auditorium

J.W.B. Stark and R.J. Stark, *Analysis and design of composite steel and concrete structures for buildings according to Eurocode 4* (Steel Design 4), Zoetermeer 2022, ISBN 978-90-75146-06-6, format 23x25 cm, 228 p.

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Joints

Steel Design 5



CONTENT

- 1 Joint characteristics in frames
- 2 Pinned joints in frames
- 3 Moment-resistant joints in frames
- 4 Column bases
- 5 Hollow section joints

J.W.B. Stark and J. Wardenier, *Analysis and design of bolted and welded joints in steel frames and in tubular structures according to Eurocode 3* (Steel Design 5), Zoetermeer 2023, ISBN 978-90-75146-08-0, format 23x25 cm, 252 p.

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Structural safety

The safety of structures – of buildings in which we live and work amongst others – is a fundamental need of humanity. In many countries, the government sees it as its responsibility to guarantee structural safety. Structural safety is mostly addressed by legal regulations which designate the building standards and codes to be used, in particular the Eurocodes. In this way minimum requirements for the safety of structures are assured. Buildings and bridges (or parts of these) can collapse when their structural elements do not satisfy these minimum requirements, leading to significant damage or even casualties (fig. 1.1 and 1.2). Many publications, amongst others [1], discuss structural safety and reliability.

The general principles of structural safety are presented in the basic Eurocode EN 1990. This code describes the principles of structural design and analysis and provides guidelines for inter-dependent aspects of the structural reliability.

This chapter first discusses the theory and background of EN 1990 regarding:

- probability of failure (safety);
- reliability principles (taking uncertainties into account);
- design value of resistance (strength of a structure);
- design value of actions (action types and combinations of actions);
- reliability (consequence classes and reliability index).

Finally, the content and structure of EN 1990 is discussed briefly, following the order of the chapters in the code.

1.1 Collapsed parking deck at a hotel in Tiel (The Netherlands, 2002) due to, amongst other things, insufficient stability of the edge beam.



1.2 Collapsed Saint Anthony Falls Bridge in Minneapolis (USA, 2007) due to incorrectly designed joints (gusset plates) in the truss.



1.1 Probability of failure

When designing a structure, the structural engineer needs to show that the effect of actions E on the structure is lower than the resistance R of the structure during its design working life. The term 'actions' is broad, covering not only loads but also, for example, imposed deformations, and expansion due to changing temperature and creep.

The effect of actions on a structure depends on the following basic variables:

- actions and environmental influences;
- material and product properties;
- geometrical properties of the structure and its elements.

Using applied mechanics the effect of actions can be described in terms of internal forces – such as bending moments, shear forces and normal forces – or, for example, stresses, strains or deflections. After determining the geometrical properties of the structure – including the cross-section dimensions – the cross-section properties can be determined from books of tables, and the magnitude of the actions is determined using the different parts of the Eurocode on actions, EN 1991, see *Structural basics 2* (Actions and deformations). The material properties for steel structures follow from EN 1993-1-1. Thus, using this approach, all basic variables get one specific value.

The assessment procedure for structures is in this way similar as a deterministic approach. The structural engineer should appreciate that many basic variables in reality do not have the exact same values as those applied in the analysis. This is due to the fact that all basic variables are, statistically speaking, so-called stochastic variables: actions vary in time, dimensions vary between tolerance limits and material properties show certain variability. The structural engineer should therefore show that the probability of failure of the structure is sufficiently small. By following the Eurocode approach the engineer will implicitly ensure that the probability of failure is sufficiently small.

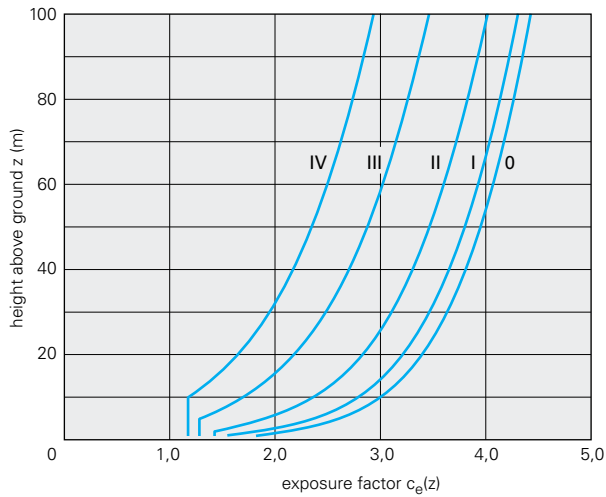
The probability of failure of a structure is denoted as P_f . The probability of survival P_s is the probability that the structure does not fail and is complementary to the probability of failure. According to the theory of probability, the sum of the probability of failure and the probability of survival is equal to one. The probability of survival is referred to as the reliability. The reliability of the structure is then:

$$P_s = 1 - P_f \quad (1.1)$$

The result of a reliability analysis is the probability that the structure survives, which is known as its reliability. This probability of survival is generally almost equal to 1, for example in the order of $P_s = 0,999999$, where the number 1 is equal to 100%. This can also be written as:

$$P_s = 0,999999 = 1 - 0,000001 = 1 - 10^{-6} \quad (1.2)$$

Equation (1.2) shows that the probability of failure is $P_f = 10^{-6}$. In the interests of clarity the result of a reliability analysis, although defining reliability, is usually presented as a probability of failure.



2.20 Exposure factor $c_e(z)$ for $c_o(z) = 1,0$ and $k_1 = 1,0$ as a function of the height above terrain and of terrain category.

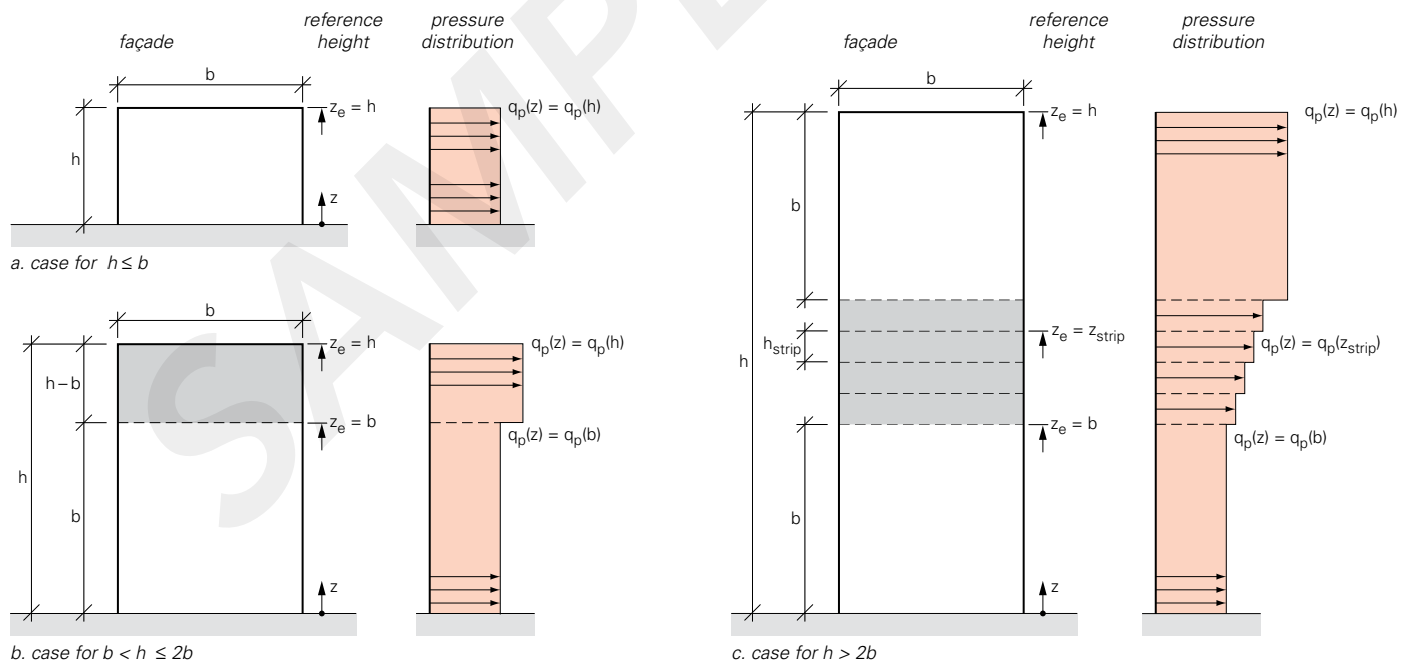
For flat terrain where $c_o(z) = 1,0$, the exposure factor $c_e(z)$ is shown in figure 2.20 as a function of the height and terrain category as defined in table 2.19.

Wind pressures and wind forces

Wind actions on structures and elements of structures shall be determined taking external and internal pressures into account. A distinction is made between wind pressures and wind forces. Wind pressures are used for the design of façades of buildings, façade cladding, connections and structural parts. Wind forces are used for design considering overall wind effects on structural components, billboards, cylinders and spheres, trusses, scaffolding and flags. EN 1991-1-4 therefore contains different aerodynamic coefficients for structures.

- **Internal and external pressure coefficients:** used for buildings and circular cylinders. External pressure coefficients are divided into overall coefficients for areas larger than 10 m^2 and local coefficients for small elements and connections.
- **Net pressure coefficients:** used for canopies and free-standing walls, parapets and fences.
- **Friction coefficients:** used for walls and surfaces.
- **Force coefficients:** used for signboards, structural elements with rectangular, sharp edged or regular polygonal sections, circular cylinders, spheres, trusses (lattice structures) and scaffoldings and flags.

2.21 Reference height z_e , depending on the building dimensions corresponding velocity pressure profile $q_p(z)$ for façades.



Wind pressure

The external wind pressure w_e acting on the external surfaces, EN 1991-1-4, cl. 5.2(1), should be obtained from:

$$w_e = q_p(z_e) \cdot c_{pe} \quad (2.19)$$

Where:

$q_p(z_e)$ peak velocity pressure at the reference height z_e ;

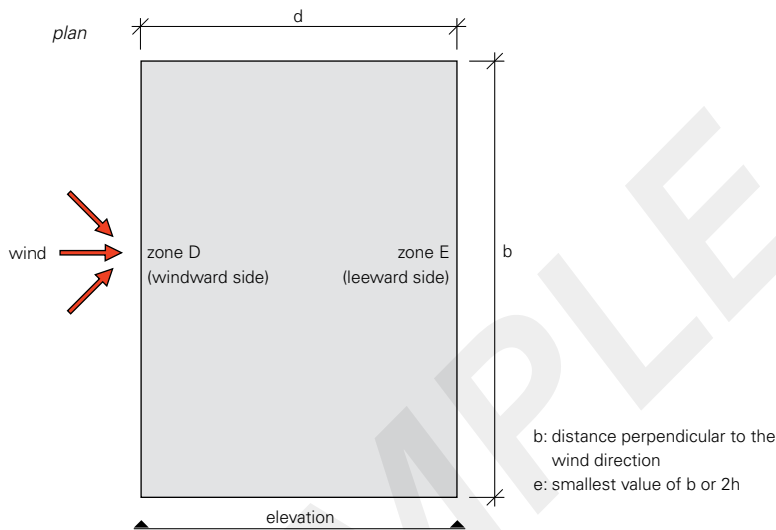
z_e reference height for the external wind pressure;

NA c_{pe} external pressure coefficient.

The reference height z_e for external wind pressure depends on the dimensions of the building according to figure 2.21. Figure 2.22 provides the external pressure coefficients c_{pe} in several

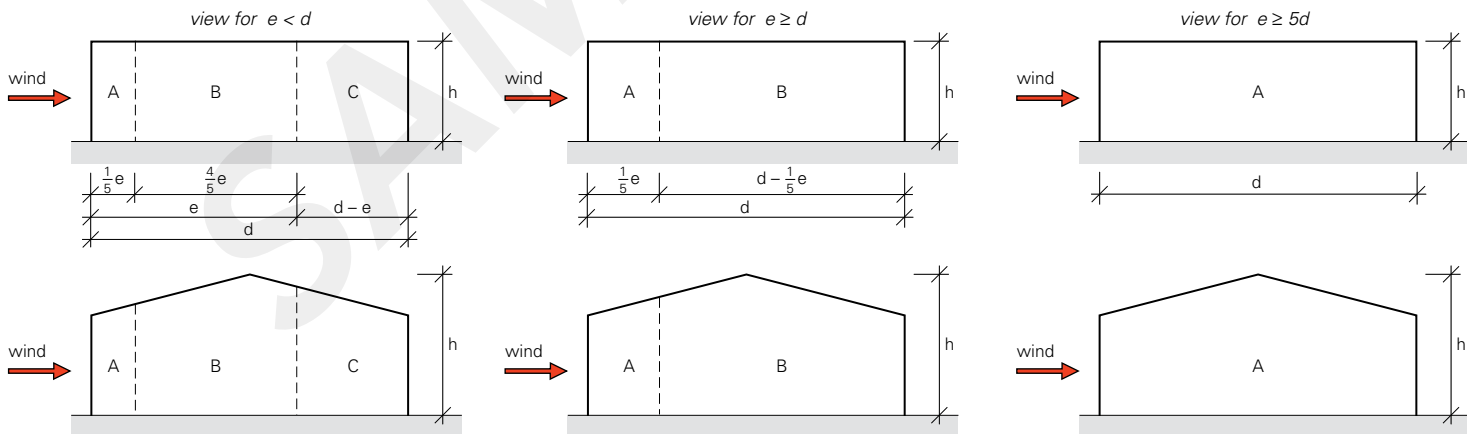
2.22 External pressure coefficient c_{pe} for the different zones of vertical façades of buildings with a rectangular plan.

For $h/d > 5$, EN 1991-1-4, cl. 7.6 till 7.8 and cl. 79.2 should be applied.



		h/d		
		≤ 0,25	1	5
zone A	$c_{pe,1}$	-1,4	-1,4	-1,4
	$c_{pe,10}$	-1,2	-1,2	-1,2
zone B	$c_{pe,1}$	-1,1	-1,1	-1,1
	$c_{pe,10}$	-0,8	-0,8	-0,8
zone C	$c_{pe,1}$	-0,5	-0,5	-0,5
	$c_{pe,10}$	-0,5	-0,5	-0,5
zone D	$c_{pe,1}$	+1,0	+1,0	+1,0
	$c_{pe,10}$	+0,7	+0,8	+0,8
zone E	$c_{pe,1}$	-0,3	-0,5	-0,7
	$c_{pe,10}$	-0,3	-0,5	-0,7

linear interpolation is allowed for in-between values



Example 3.1

- **Given.** A T-section formed from half a HEB 300 section (fig. 3.19).
- **Question.** Determine shape factor α_y for the y-axis.
- **Answer.** If it is assumed that the plastic neutral axis is located in the flange of the section, the height of the plastic zone under the plastic neutral axis $e_{z,pl}$ is (fig. 3.19):

$$e_{z,pl}b = \frac{1}{2}A$$

Rewritten and calculated:

$$e_{z,pl} = \frac{A}{2b} = \frac{7454}{2 \cdot 300} = 12,4 \text{ mm}$$

The distance z_1 of the plastic neutral axis to the centre of gravity Z_1 of the part of the section above the plastic neutral axis – neglecting the root radii (fillets) – is:

$$\begin{aligned} z_1 &= \frac{0,5b(t_f - e_{z,pl})^2 + (h - t_f)t_w(t_f - e_{z,pl} + 0,5(h - t_f))}{0,5A} \\ &= \frac{0,5 \cdot 300 \cdot (19 - 12,4)^2 + (150 - 19) \cdot 11 \cdot (19 - 12,4 + 0,5 \cdot (150 - 19))}{0,5 \cdot 7455} = 29,6 \text{ mm} \end{aligned}$$

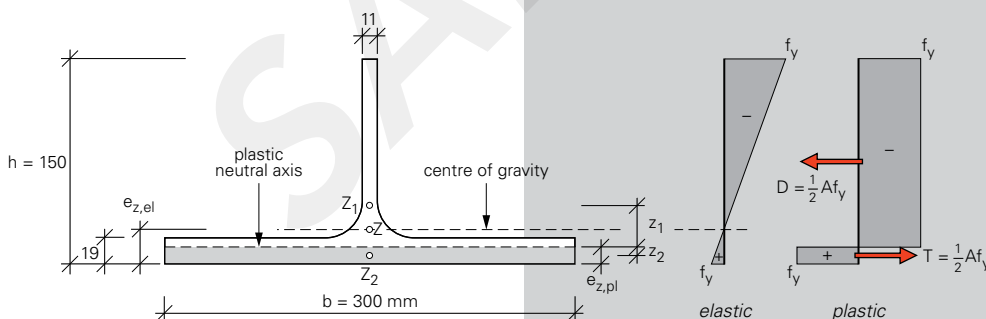
The distance z_2 of the plastic neutral axis to the centre of gravity Z_2 of the part of the section below the plastic neutral axis is:

$$z_2 = 0,5e_{z,pl} = 0,5 \cdot 12,4 = 6,2 \text{ mm}$$

The plastic section modulus $W_{pl,y}$ is the sum of the first moments of area of both parts of the section with respect to the plastic neutral axis:

$$W_{pl,y} = S_1 + S_2 = \frac{1}{2}A(z_1 + z_2) = \frac{1}{2} \cdot 7455 \cdot (29,6 + 6,2) = 133 \cdot 10^3 \text{ mm}^3$$

3.19 T-section formed from half a HEB 300 section.



If the root radii are taken into account, the plastic section modulus is $W_{pl,y} = 137 \cdot 10^3 \text{ mm}^3$ and the elastic section modulus $W_{el,y} = 69,6 \cdot 10^3 \text{ mm}^3$. Then the shape factor is:

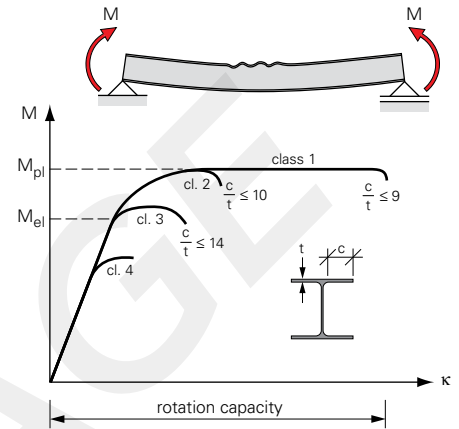
$$\alpha_y = \frac{W_{pl,y}}{W_{el,y}} = \frac{137 \cdot 10^3}{69,6 \cdot 10^3} = 1,97$$

3.2.2 Classification

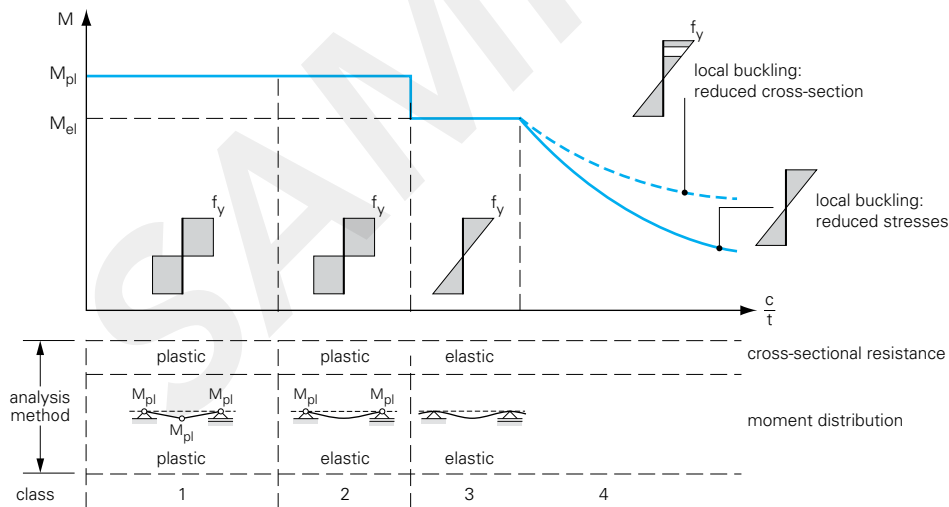
The properties of sections loaded in combined compression and bending depend to a large extent on the c/t ratios, where c is the width and t is the thickness, of the compressed plates forming the section. This can be seen from the schematic M/κ diagrams of an I-section loaded in bending, shown in figure 3.20. The shape of the M/κ diagram depends on the c/t ratio of the flange in compression, where c is measured from the root radius to the end of the flange. The load bearing capacity of the flange and the curvature at failure – thus also the rotation capacity – increase when the flange is stockier, that is has a greater relative thickness. This behaviour depends on to what extent the compression flange is susceptible to local buckling.

When choosing a global analysis method, the structural engineer must anticipate the cross-sectional deformation behaviour. For example, plastic theory may only be used when the sections have sufficient deformation capacity so that the assumed redistribution of moments can occur. This means that the rotation capacity – more generally the deformation capacity – has to be sufficiently large. Also, a distinction can be made between sections where local buckling occurs in the elastic range and sections where local buckling occurs in the plastic part of the behavioural response. For the former, a local buckling analysis is required; for the latter this is not necessary.

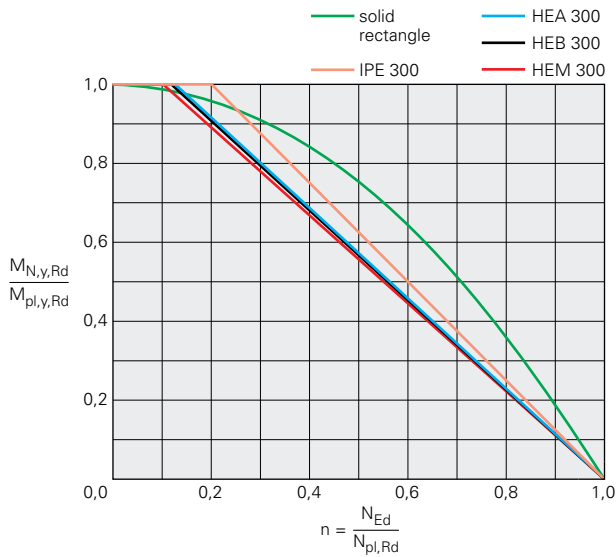
Comparing the deformation capacity of a cross-section with the deformation capacity required in a given situation is too complicated for everyday design. Therefore the Eurocode uses a classification system whereby cross-sections are subdivided into one of four classes depending on the plate dimensions and the yield stress (EN 1993-1-1, cl. 5.5). The global analysis methods, which are admissible for the different cross-section classes, are shown schematically in figure 3.21. The moment resistance, which can be achieved for pure bending, is also shown in this figure. The four cross-section classes are described below, each with a corresponding analysis method.



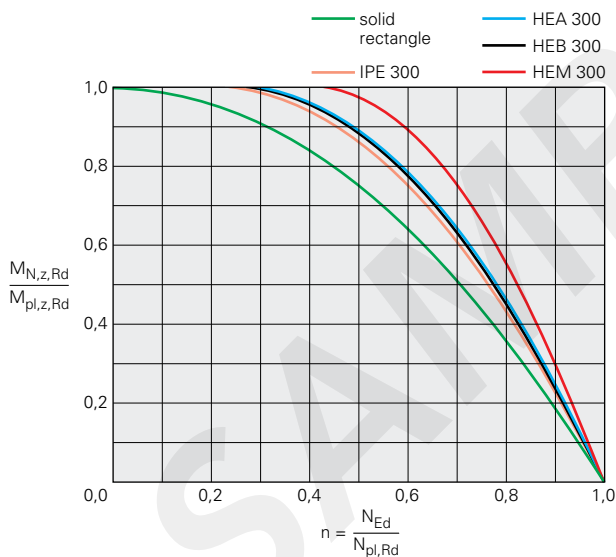
3.20 Influence of the c/t ratio of the flange of a rolled I-section in S235 on the M/κ diagram.



3.21 Relationship between cross-section class and analysis methods.



7.12 Relative reduced design bending moment resistance with respect to the y-axis as a function of the utilization ratio for axial force.



7.13 Relative reduced design bending moment resistance with respect to the z-axis as a function of the utilization ratio for axial force.

Figure 7.12 shows, for several sections, the relative reduced design bending moment resistance with respect to the y-axis allowing for axial force ($M_{N,y,Rd}/M_{pl,y,Rd}$), as a function of the utilization ratio for axial force n . Figure 7.13 shows, for several sections, the relative reduced design bending moment resistance with respect to the z-axis allowing for axial force ($M_{N,z,Rd}/M_{pl,z,Rd}$), as a function of the utilization ratio for axial force n .

- For rectangular hollow sections with a uniform wall thickness:

$$M_{N,y,Rd} = M_{pl,y,Rd} \frac{1-n}{1-0,5a_w} \leq M_{pl,y,Rd} \quad (7.50)$$

$$M_{N,z,Rd} = M_{pl,z,Rd} \frac{1-n}{1-0,5a_f} \leq M_{pl,z,Rd}$$

with

$$\text{for rolled hollow sections: } a_w = \frac{A-2bt}{A} \leq 0,5 \quad a_f = \frac{A-2ht}{A} \leq 0,5$$

$$\text{for welded hollow sections: } a_w = \frac{A-2bt_f}{A} \leq 0,5 \quad a_f = \frac{A-2ht_w}{A} \leq 0,5$$

- For circular hollow sections with a uniform wall thickness:

$$M_{N,y,Rd} = M_{N,z,Rd} = M_{pl,Rd} (1-n^{1,7}) \quad (7.51)$$

For bi-axial bending, the following design rule needs to be satisfied:

$$\left(\frac{M_{y,Ed}}{M_{N,y,Rd}} \right)^\alpha + \left(\frac{M_{z,Ed}}{M_{N,z,Rd}} \right)^\beta \leq 1,0 \quad (7.52)$$

In equation (7.52), α and β are constants. Conservatively, it may be assumed that $\alpha = \beta = 1,0$. More accurately, these constants may be determined as:

- I- and H- sections $\alpha = 2$ and $\beta = 5n \geq 1$;
- circular hollow sections $\alpha = \beta = 2$;
- rectangular hollow sections $\alpha = \beta = \frac{1,66}{1-1,13n^2} \leq 6$.

Figure 7.14 shows the interaction under biaxial bending for several sections and utilization ratios.

In EN 1993-1-1, cl. 6.2.9.1, also the interaction equation for the combination of bending and axial force for circular hollow sections in class 1 and 2 is given:

$$\frac{M_{Ed}}{1,04M_{c,Rd}} + \left(\frac{N_{Ed}}{N_{c,Rd}} \right)^{1,7} \leq 1,0 \quad \text{and} \quad \frac{M_{Ed}}{M_{c,Rd}} \leq 1,0 \quad (7.53)$$

Cross-sectional class 3

EN 1993-1-1, cl. 6.2.9.2 gives a design rule for bending and axial force acting in combination on a cross-section in class 3. For these sections, the design rule consists simply of checking the maximum normal stress in the axial direction:

$$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}} \Rightarrow \frac{\sigma_{x,Ed}}{f_y/\gamma_{M0}} \leq 1,0 \quad (7.54)$$

Where $\sigma_{x,Ed}$ is the design normal stress in the axial direction due to the bending moment and axial force, taking bolt hole reduction into account.

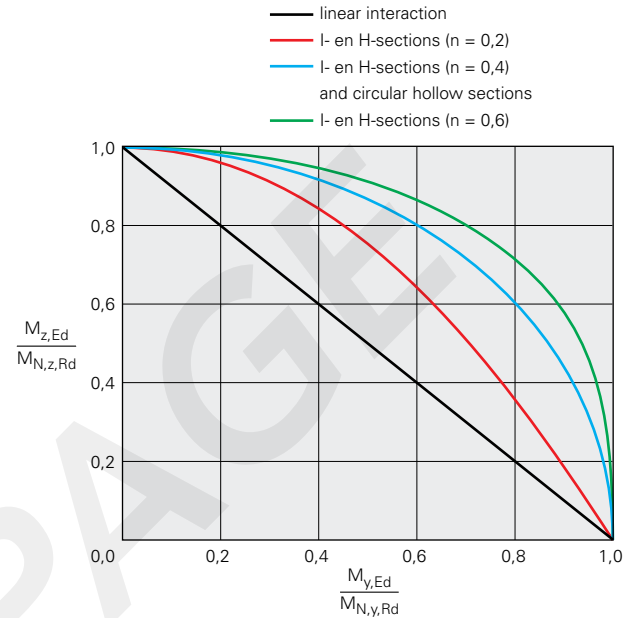
Cross-section class 4

EN 1993-1-1, cl. 6.2.9.3 treats the design for bending and axial force acting in combination on cross-sections in class 4. The same principle applies to these sections as is adopted for cross-sections in class 3. The difference however is that for class 4 cross-sections, the design normal stress in the axial direction $\sigma_{x,Ed}$ should be based on the effective cross-section. As an alternative to equation (7.54), equation (7.55) can be applied for the effective cross-section. The shift of the center of gravity of the effective cross-section, relative to the center of gravity of the gross cross-section, should be taken into account (fig. 7.15). In this case, the following design rule applies:

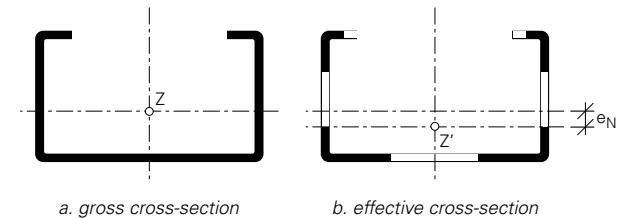
$$\frac{N_{Ed}}{A_{eff} f_y / \gamma_{M0}} + \frac{M_{y,Ed} + N_{Ed} e_{N,y}}{W_{eff,y,min} f_y / \gamma_{M0}} + \frac{M_{z,Ed} + N_{Ed} e_{N,z}}{W_{eff,z,min} f_y / \gamma_{M0}} \leq 1,0 \quad (7.55)$$

Where:

- A_{eff} effective area of the cross-section under uniform compression;
- $W_{eff,min}$ effective section modulus (based on the outermost fiber with the largest elastic stress, which is where the yield stress is reached first) of the cross-section for pure bending about the axis considered;
- e_N shift of the center of gravity with respect to the axis considered when the cross-section is subjected to axial force alone.



7.14 Interaction curves for bi-axial bending.



7.15 Shift of the center of gravity for cross-section class 4.

The coating can be applied either off-site, in the fabrication shop, or on the construction site. With off-site application any damage that occurs during transportation will need to be repaired, and there will need to be on-site treatment of the joints. Also, attention to the thickness of the coating must be paid, because it affects drying time in the fabrication shop.

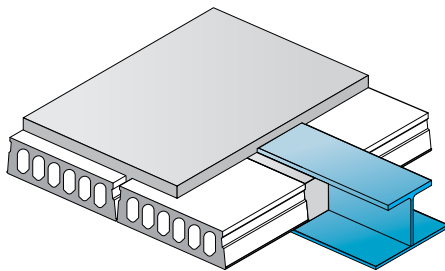
Most intumescent coatings are suitable for use in a non-aggressive internal environment. Some are suitable for outdoor applications. Approved test reports based on a standardized testing method^[11] play a very important role in the approval by regulatory authorities and the fire department. Also, quality assurance during the application of the coating, including controlling the application conditions and checking the applied layer thickness, is important. Specific guidance on quality assurance is available, see [2].

NA

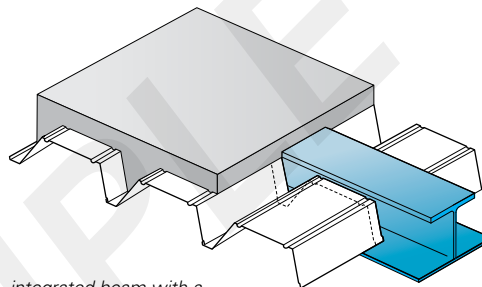
Structural integration

The designer can choose to integrate steel columns partially or totally within the depth of a façade or separating wall (fig. 1.16). This is also true for bracing members and other means of providing stability. The floor structure can also be designed such that the steel floor beams are located within the depth of the floor.

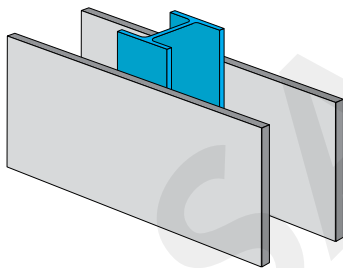
1.16 Integrated steel in floor and wall structures.



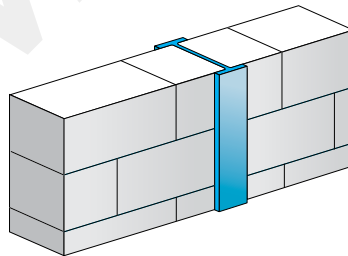
integrated beam with a hollow core slab



integrated beam with a deep composite slab



column in a (metal stud) wall



column in a masonry wall



1.17 Integration of a steel column in a timber framed façade (covered with gypsum board on the inner side). Without additional measures, the fire resistance of the column is at least 30 minutes.

The costs of additional fire resistance measures are relatively low for integrated steel structures. Choosing such a solution does, however, have other design consequences. Also, the steel structure itself is not exposed so its aesthetic benefits can not be used (fig. 1.17).

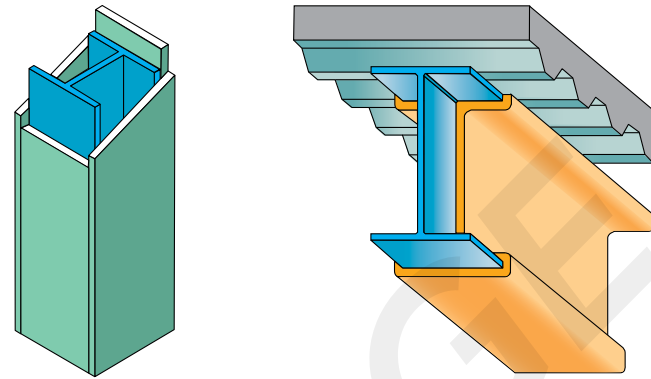
An integrated steel structure always has a fire resistance of 30 minutes and sometimes even 60 minutes. A fire resistance of 120 minutes can be achieved with limited additional measures. The fire resistance of integrated beams can be calculated relatively easy, see *Fire 2* (Calculation of fire resistance), section 2.8.

Heat-insulating protection

Over-designing, i.e. using bigger members than needed at room temperature, is not a suitable option for a fire resistance of 60 minutes or more. The steel temperature after 60 minutes of fire is more than 900 °C, which leads to a strength reduction of 5% or less. However the heating of the steel structure can be slowed down significantly by application of heat-insulating protection in the form of a hollow encasement by suitable boards fixed around the section, or by sprayed mortar as a contour encasement (fig. 1.18, 1.19 and 1.20; see also fig. 1.15).

When insulation by boards or sprayed mortar is applied it is often not necessary to consult the structural engineer. The thickness of the protection can be determined based on information provided by the manufacturer, derived from tests^[10]. This is based on safe values for the critical steel temperature. However, such a solution does not normally result in the most economical solution, so it is useful to consult a structural engineer. The manufacturer(s) can then determine the most economic thickness of the material, based on information from the engineer. If information about the critical steel temperatures is already included in the contract documentation, the contractor can choose the most suitable solution.

With heat-insulating protection a fire resistance of 120 minutes can be achieved. The fire resistance of a protected steel section can be calculated relatively easily, see *Fire 2*, section 2.5 (with example 2.3), 2.2.4 and 2.9.



1.18 Heat-insulating protection: board encasement around a section and sprayed mortar as a contour encasement.



1.19 Protection of the bottom flange of an integrated beam with mineral wool boards. With a minimum plate thickness often a fire resistance of 120 minutes can be achieved.



1.20 Application of a heat-insulating sprayed cementitious mortar layer. Any level of fire resistance can be achieved with this method.

The degree of utilization μ_0 is always less than or equal to the reduction factor for the the design load level in the fire situation $\eta_{fi} = E_{fi,d}/E_d$. The reduction factor η_{fi} is therefore an upper limit approach to the degree of utilization of μ_0 . Take as an example a floor beam in an office building with combination factor $\psi_2 = 0,3$, loaded by a permanent load G_k and a variable load Q_k . If the beam has been designed on strength and meets the requirements at normal temperature (unity check of $E_d/R_{fi,d,0}$ less than or equal to 1,0), then $E_d = R_{fi,d,0}$ and therefore also $\mu_0 = \eta_{fi}$. This yields to the equation:

$$\eta_{fi} = \frac{G_k + \psi_2 Q_k}{\gamma_G G_k + \gamma_Q Q_k} = \frac{G_k + 0,3 Q_k}{1,2 G_k + 1,5 Q_k} \geq \mu_0 \quad (2.3)$$

The partial load factors are used according to EN 1990: $\gamma_G = 1,2$ for the permanent and $\gamma_Q = 1,5$ for the variable action (dependant on the National Annex). Depending on the loads, for offices the reduction factor for the the design load level in the fire situation varies usually between $\eta_{fi} = 0,48$ ($G_k = Q_k$) and $\eta_{fi} = 0,59$ ($G_k = 2Q_k$). Equation (2.3) is shown graphically in figure 2.3 for different occupancies (different values of ψ_2). See also table 4.3 in *Fire 4*. For countries in which the combination factor ψ_1 for frequent actions is mandatory instead of the combination factor ψ_2 for quasi-permanent actions, the equation has to be slightly modified.

For centrally loaded columns in braced frames with n storeys, the upper limit for the degree of utilization in offices (with $\psi_2 = 0,3$ for fire and $\psi_0 = 0,5$ for normal load combinations (in which two storeys are fully loaded and $n - 2$ stories with combination factor ψ_0)) is set at:

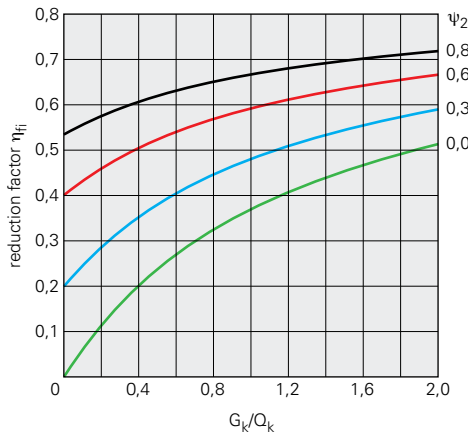
$$\eta_{fi} = \frac{n G_k + n \psi_2 Q_k}{n 1,2 G_k + 2 \cdot 1,5 Q_k + (n - 2) \cdot 1,5 \psi_0 Q_k} = \frac{G_k + 0,3 Q_k}{1,2 G_k + 0,75 Q_k \left(1 + \frac{2}{n}\right)} \geq \mu_0 \quad (2.4)$$

For two storeys, the equations (2.3) and (2.4) are the same. For three or more storeys, the degree of utilization for columns is higher than that for beams. This is because the fundamental combination of actions for columns at normal temperature consists of two storeys fully loaded with a variable load, which is reduced on the other storeys by the combination factor ψ_0 . The effect of the factor ψ therefore decreases with an increasing number of storeys and is less for columns than for beams.

In practice, the degree of utilization is usually in the order of 0,5. This value is smaller than follows from equation (2.3) or equation (2.4). This is because there is often extra loadbearing capacity available, for example in the case of beams because the deflection criterion is decisive, or in the case of columns because for practical reasons the same profile is used at several storeys. In the event of a fire, a reduced buckling length of the columns can often be applied (see section 2.6).



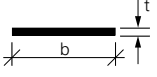

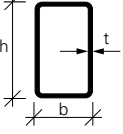
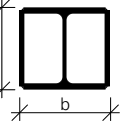
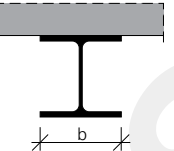
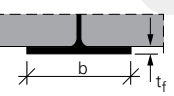
2.1.4 Section factor

The section factor takes into account the influence of the geometry of the steel section on the heating. The section factor is defined as the heated area A (in m^2 per metre of length) divided by the volume of the steel section V (in m^3 per metre of length), using m^{-1} as the unit. In practical terms, the section factor is therefore equal to the heat-exposed circumference (in m) divided by the area of the steel cross-section (in m^2).


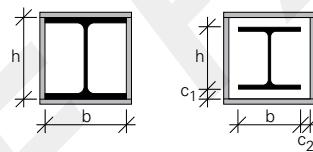
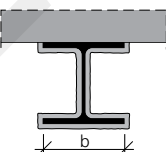
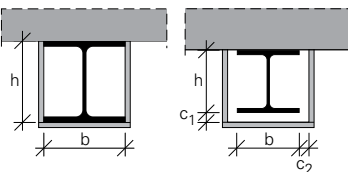


2.3 Relationship between the reduction factor for the design load level in a fire situation η_{fi} and the ratio between the permanent and the variable load G_k/Q_k for different values of the combination factor ψ_2 .

- For *unprotected* sections, the section factor is referred to as A_m/V (index m for member), where A_m is the heated circumference of the profile and V is the area of the steel cross-section (fig. 2.4).
- In the case of *protected* sections, the section factor is referred to as A_p/V (index p for 'protected'), where A_p is the inner circumference of the *protection* and V is the area of the steel cross-section (fig. 2.5a). For *protection* that is not placed directly against the section – e.g. by means of spacers or a framework – no larger circumference is used and the smallest circumference around the heated section of the profile applies (fig. 2.5b). In the case of a *protected* steel girder below a concrete floor, the circumference of the steel section must be reduced by the width of the top flange. The concrete floor protects the upper flange against heating (fig. 2.5c and d).

cross-section	description	section factor A_m/V
	open section exposed to fire on all sides	$\frac{\text{section perimeter}}{\text{cross-section area}}$
	angle exposed to fire on all sides	$\frac{2}{t}$
	flat plate exposed to fire on all sides	$\frac{2(b+t)}{bt} \approx \frac{1}{t}$ (for $t \ll b$)
	circular hollow section exposed to fire on all sides	$\frac{1}{t}$
	rectangular hollow section – or welded box of uniform thickness – exposed to fire on all sides	$\frac{2(b+h)}{\text{cross-section area}}$ $\approx \frac{1}{t}$ (for $t \ll b$)
	I-section with box reinforcement, exposed to fire on all sides	$\frac{2(b+h)}{\text{cross-section area}}$
	open section exposed to fire on three sides	$\frac{\text{section perimeter} - b}{\text{cross-section area}}$
	I-section flange (or a flat plate) exposed to fire on three sides	$\frac{b+2t_f}{bt_f} \approx \frac{1}{t_f}$ (for $t_f \ll b$)

2.4 Section factor A_m/V (without 'shadow effect') for unprotected sections.

cross-section	description	section factor A_p/V
a 	contour encasement of uniform thickness, exposed to fire on all sides	$\frac{\text{section perimeter}}{\text{cross-section area}}$
b 	hollow encasement of uniform thickness ^[1] , exposed to fire on all sides	$\frac{2(b+h)}{\text{cross-section area}}$
c 	contour encasement of uniform thickness, exposed to fire on three sides	$\frac{\text{section perimeter} - b}{\text{cross-section area}}$
d 	hollow encasement of uniform thickness ^[1] , exposed to fire on three sides	$\frac{2h+b}{\text{cross-section area}}$

1. The clearance dimensions c_1 and c_2 should not normally exceed $h/4$.

2.5 Section factor A_p/V for protected sections.

2.2.1 Plastic moment resistance in sagging bending; longitudinal shear not critical

This section considers the plastic moment resistance $M_{pl,Rd}$ for sagging bending, assuming that the longitudinal shear connection is not critical (full shear connection). The calculation of the required number of shear connectors to assure this is considered in section 2.3.3, and the consequences of applying fewer connectors (partial shear connection) in section 2.3.4. The following types of steel sections are considered:

- symmetrical sections made of grade S355 steel or lower;
- asymmetric sections;
- sections of grade S420 or S460 steel.

Symmetrical sections made of grade S355 steel or lower

The starting point is to assume that there will be sufficient shear connectors to fully exploit the strength of the steel or concrete (whichever is the lower). This means that the longitudinal shear connection will not be critical. Additionally the following five assumptions are made:

- there is full interaction between the structural steel, reinforcement and concrete;
- a plastic stress block (f_{yd}) acts over the entire structural steel cross-section, in tension as well as in compression;
- the slab reinforcement yields with a stress f_{sd} whether in tension or compression; alternatively, reinforcement may be neglected in the compression zone of the concrete slab;
- the distribution of compression stresses in the concrete is uniform and equal to $0,85f_{cd}$ over the full depth of the slab, between the plastic neutral axis and outer fibre;
- any contribution of profiled steel sheeting in the case of a composite slab is completely ignored.

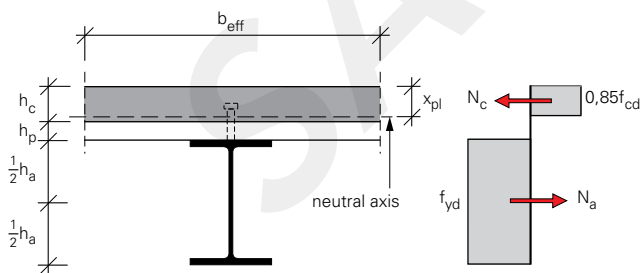
The resistance of a cross-section in sagging depends on the location of the plastic neutral axis.

The neutral axis might be positioned in:

- the concrete slab;
- the upper flange of the steel section;
- the web of the steel section.

2.14 Stress distribution with the plastic neutral axis in the concrete slab (case 1).

The equations given below have been established for a composite beam with a composite slab. However, they also apply to solid slabs, in which case $h_p = 0$.

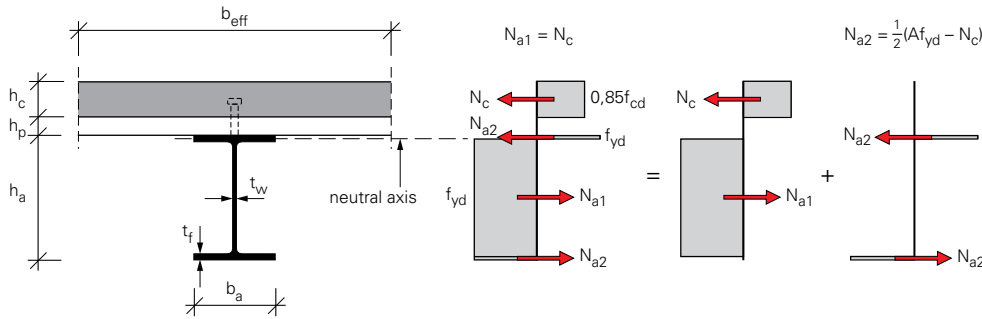


- **Case 1. Neutral axis in the concrete slab.** This situation occurs when the axial resistance of the steel beam is less than that of the concrete slab, or $Af_{yd} < h_c b_{eff} 0,85f_{cd}$ (fig. 2.14). From the force balance $N_c = N_a$ it follows that $x_{pl} b_{eff} 0,85f_{cd} = Af_{yd}$, therefore:

$$x_{pl} = \frac{Af_{yd}}{b_{eff} 0,85f_{cd}}$$

From a consideration of moment equilibrium:

$$M_{pl,Rd} = Af_{yd} \left(\frac{1}{2} h_a + h_p + h_c - \frac{1}{2} x_{pl} \right) \quad (2.9)$$



2.15 Stress distribution with the plastic neutral axis in the flange of the steel section (case 2).

- **Case 2. Neutral axis in the upper steel flange.** This situation occurs when the resistance of the concrete slab is less than that of the steel beam, but greater than that of the steel web alone, so $Af_{yd} > h_c b_{eff} 0,85f_{cd} > (h_a - 2t_w)t_w f_{yd}$ (fig. 2.15). The moment resistance can be divided into two components. One component follows from the internal balance between the force in the concrete slab and that in the web and part of the flanges of the steel beam. The other component is the result of the internal equilibrium of the remaining parts of the flanges:

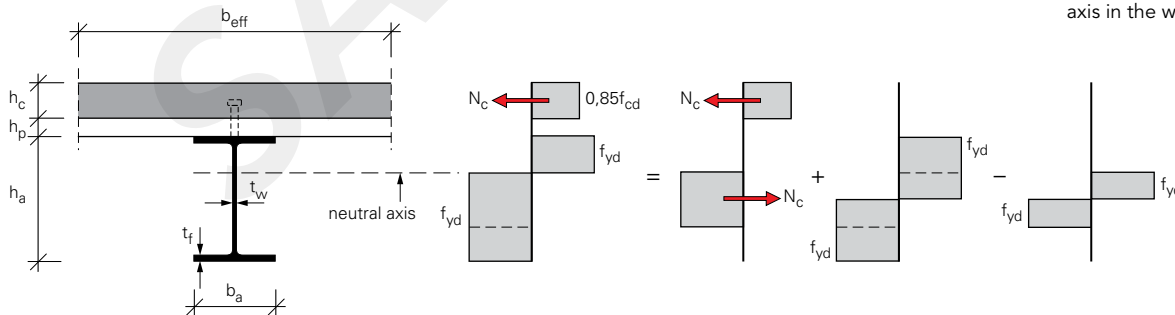
$$M_{pl,Rd} = N_c \left(\frac{1}{2}h_a + h_p + \frac{1}{2}h_c \right) + \frac{1}{2} \left(Af_{yd} - N_c \right) \left(h_a - \frac{Af_{yd} - N_c}{2b_a f_{yd}} \right) \quad (2.10)$$

$$N_c = h_c b_{eff} 0,85f_{cd}$$

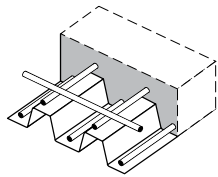
- **Case 3. Neutral axis in the web of the steel section.** This situation occurs when the resistance of the concrete slab is less than that of the web alone of the steel beam, i.e. $h_c b_{eff} 0,85f_{cd} < (h_a - 2t_w)t_w f_{yd}$ (fig. 2.16). In this case the moment resistance can be divided into three components:

$$M_{pl,Rd} = N_c \left(\frac{1}{2}h_a + h_p + \frac{1}{2}h_c \right) + W_{apl} f_{yd} - \frac{N_c^2}{4t_w f_{yd}} \quad (2.11)$$

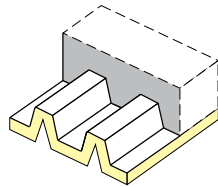
$$N_c = h_c b_{eff} 0,85f_{cd}$$



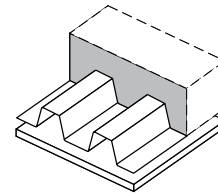
2.16 Stress distribution with the plastic neutral axis in the web of the steel section (case 3).



additional reinforcement



insulating coating



fire proof ceiling

3.6 Fire limit state

Composite slabs should be assessed against the following criteria in case of fire:

- mechanical resistance (criterion R);
- thermal insulation (criterion I);
- integrity (criterion E);

The first criterion refers to the resistance of the slab, and the second and third criteria to its separating function (compartmentation).

A composite slab with normal weight concrete that has been designed for room temperature conditions according to EN 1994-1-1 may be assumed to achieve a fire resistance of 30 minutes. This has been proven by tests, so there is no need to determine the fire resistance by means of a test or explicit calculations. When lightweight concrete is used, or when there is a requirement for a fire resistance of 60 minutes or more, testing or additional calculations are necessary. In general, therefore, additional provisions are needed to achieve the required level of fire resistance (fig. 3.32).

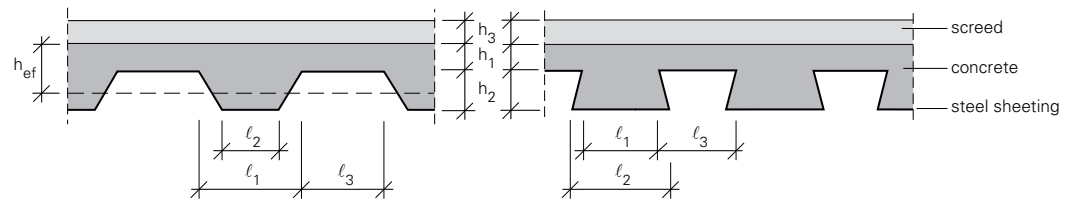
EN 1994-1-2, cl. 4.3 provides rules for the assessments of both *unprotected* and *protected* composite slabs. For slabs within the scope of application of EN 1994-1-2 it may be assumed that the composite slab meets criterion E (including flame density). EN 1994-1-2, annex D contains models for the calculation of fire resistance with respect to criterion I (thermal insulation) and criterion R (mechanical resistance) of unprotected composite slabs. Both models are discussed below.

3.6.1 Unprotected composite slabs

The geometric scope of the rules for the assessment of unprotected composite slabs in EN 1994-1-2, annex D is shown in figure 3.33. These limits apply to both normal weight (NC) and lightweight (LC) concrete. This section deals with the following subjects;

- thermal behaviour in case of fire;
- thermal insulation (criterion I);
- mechanical behaviour in case of fire;
- mechanical resistance (criterion R).

3.33 Scope of the design rules in EN 1994-1-2, annex D.



trapezoidal deck	$80,0 \leq l_1 \leq 155,0$ mm
notation and	$32,0 \leq l_2 \leq 132,0$ mm
application limits	$40,0 \leq l_3 \leq 115,0$ mm
	$50,0 \leq h_1 \leq 125,0$ mm
	$50,0 \leq h_2 \leq 100,0$ mm

dovetail deck	$77,0 \leq l_1 \leq 135,0$ mm
notation and	$110,0 \leq l_2 \leq 150,0$ mm
application limits	$38,5 \leq l_3 \leq 97,5$ mm
	$50,0 \leq h_1 \leq 130,0$ mm
	$30,0 \leq h_2 \leq 60,0$ mm

Thermal behaviour in case of fire

In an unprotected composite slab, the steel sheeting is directly exposed to the fire so it heats up quickly. Thanks to the heat absorbing capability of the concrete the temperature in the steel sheet remains, however, significantly lower than the temperature of the fire, see lines 1 and 2 in figure 3.34. The temperature in any lower reinforcement (between lines 3 and 5) rises less rapidly than the temperature of the steel sheeting, making the reinforcement an important contributor to the mechanical resistance of the slab. Due to the shape of the steel sheeting the heat flux is mainly two-dimensional. Furthermore, the concrete type (normal weight or lightweight) and the thickness of the concrete influence the heating behaviour. Hogging reinforcement adjacent to internal supports, and the top part of the concrete, remain relatively cool (lines 6 and 7).

The temperature reached on the upper (unexposed) surface is an important aspect in the assessment of the fire resistance of a composite slab. In order to satisfy a separating function (compartmentation) in case of fire, the slab should not let flames or hot gasses through (criterion E), and the temperature of the upper surface (unexposed side) must not get too high (criterion I).

Thermal insulation (criterion I)

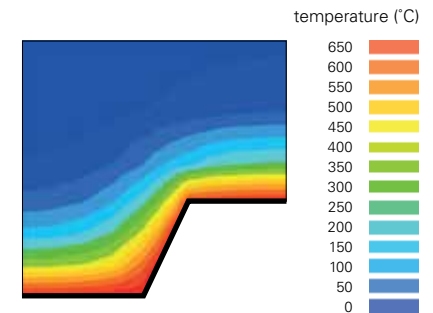
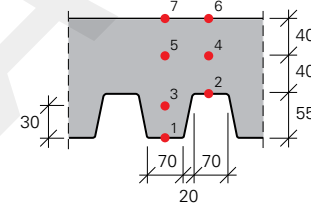
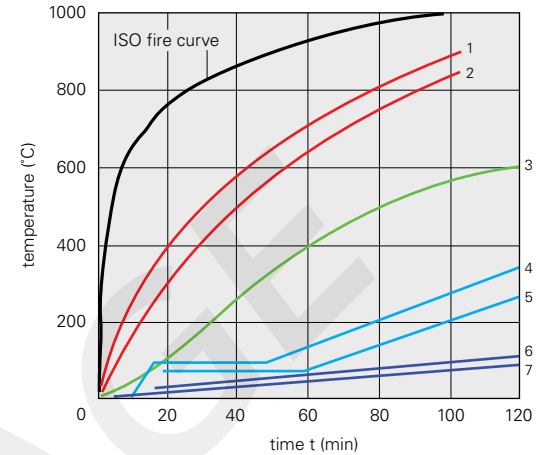
To satisfy the thermal insulation criterion, the temperature increase on the unexposed side of the slab (i.e. the upper surface) must remain less than 140 °C on average, and below a local maximum value of 180 °C. The temperature on the upper surface is not uniform due to the presence of the ribs, which result in non-uniform insulation thicknesses. Either the average temperature rise or the maximum temperature increase could therefore be critical. The fire resistance with respect to thermal insulation depends on the size and the spacing of the ribs. The simple assessment method in EN 1994-1-2, annex D makes use of two factors that are determined by geometry, namely: the view factor Φ and the geometry factor of the ribs A/L_r . Using dimensional notations as shown in figure 3.35, these factors are:

$$\Phi = \frac{\sqrt{h_2^2 + \left(\ell_3 + \frac{\ell_1 - \ell_2}{2}\right)^2} - \sqrt{h_2^2 + \left(\frac{\ell_1 - \ell_2}{2}\right)^2}}{\ell_3} \quad (3.31)$$

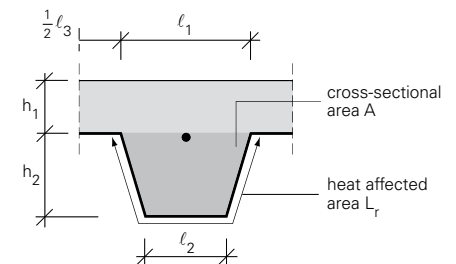
$$\frac{A}{L_r} = \frac{h_2 \left(\frac{\ell_1 + \ell_2}{2}\right)}{\ell_2 + 2 \sqrt{h_2^2 + \left(\frac{\ell_1 - \ell_2}{2}\right)^2}} \quad (3.32)$$

The fire resistance for thermal insulation t_i – the time needed for either the average temperature or the maximum temperature to reach the respective limit – can now be calculated using only one equation:

$$t_i = a_0 + a_1 h_1 + a_2 \Phi + a_3 \frac{A}{L_r} + a_4 \frac{1}{\ell_3} + a_5 \frac{A}{L_r} \cdot \frac{1}{\ell_3} \quad (\text{min}) \quad (3.33)$$



3.34 Temperature distribution in a composite slab subject to fire.

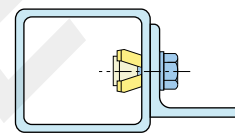


3.35 Definition of the parameters for the factor A/L_r .

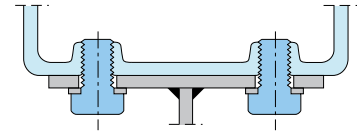
Bolts in clearance holes

Steel structures are made from sections and plates. In the fabrication shop, normally welding is used to assemble these components into columns, beams, trusses, and even portal frames. These are then transported to the construction site and assembled into a complete steel structure. Bolted connections are generally used for onsite assembly (fig. 2.1), either because the circumstances on site are not optimal for welding, or to facilitate demountability of the structure. Bolts in normal size holes (see section 2.2.6) are most common for steel structures. Other types of mechanical fasteners – in particular preloaded bolts, fitted bolts, injection bolts, rivets and pins – are treated in *Connections 3* (Slip-resistant connections, rivets, and pins). Special bolt types also exist, for example bolts that are screwed directly into the tapped wall for connections to hollow sections (fig. 2.2). Other connectors are used for joining thin gauge cold formed sections such as purlins and profiled steel sheets used for floors, roofs, and facades. Examples of these are self-tapping, thread-cutting, and self-drilling screws, and rivets and nails (fig. 2.3). Such mechanical connectors for hollow sections, cold formed sections, and thin plates are not discussed below.

2.1 Example of a bolted connection.

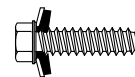


expander bolt (Lindapter)

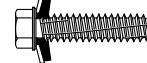


screw-thread tapped into wall (Flowdrill)

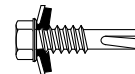
2.2 Special connectors for hollow sections.



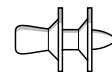
self-tapping screw \varnothing 6,3 mm
with washer and 1 mm thick elastomer
– for steel plate to be connected to main steel structure



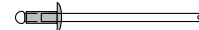
thread-cutting screw \varnothing 8 mm
with washer and with or without 1 mm thick elastomer
– for steel plate to be connected to main steel structure



self-tapping screw \varnothing 4,22-6,3 mm
– for steel plate to steel plate connections
– for fittings (without washer)
– for panels to be connected to main steel structure



shot fired nails
– for steel plate to steel structure



blind rivets \varnothing 4-6,4 mm
– for steel plate on thin walled steel lattice work
– for fittings

2.3 Connectors for cold-formed sections.

This chapter considers bolts in clearance holes (normal holes, oversized holes, and slotted holes), and the design rules used to check connections in steel structures using these bolts. The following topics are discussed:

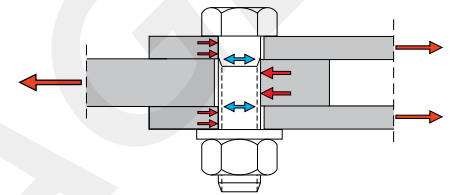
- categories of bolted connection;
- bolt characteristics;
- positioning of bolts;
- hole weakening;
- bolt resistance.

2.1 Categories of bolted connections

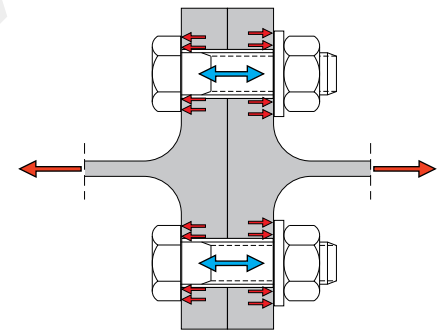
Based on the nature of the load, EN 1993-1-8, cl. 3.4 distinguishes between bolted connections loaded in shear and connections loaded in tension. Figure 2.4 shows an example of both. The necessary checks can be found in table 3.2 of EN 1993-1-8. In some cases, the bolts are loaded in both tension and shear. An example is the beam-to-column connection shown in figure 2.5.

2.1.1 Bolted connections loaded in shear

Bolts are often used with clearance holes in order to be able to insert the bolts easily: the holes are drilled larger than the diameter of the bolt itself. Using clearance holes causes that fabrication and erection deviations can be more easily accommodated. The oversize is normally 2 mm for an M20 bolt (see section 2.2.6). The clearance can cause that a connection loaded in shear deforms. Usually, the nut is tightened by hand with a wrench or with a pneumatic tool. The degree of tightening is normally based on experience, unless prescribed otherwise. The erector knows from experience not to excessively tighten the nut, to avoid the risk of fracture of the bolt.



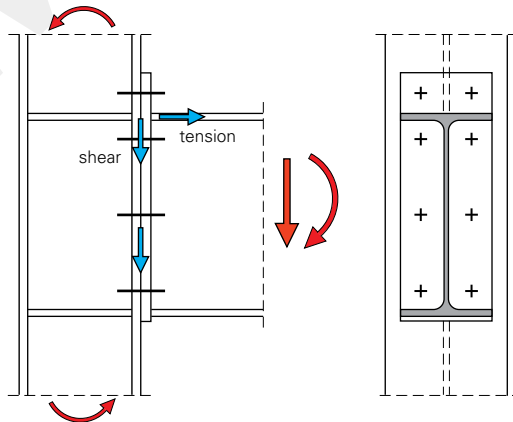
bolt loaded in shear



bolt loaded in tension

2.4 Two possible load cases for bolts.

2.5 Example of a connection with bolts loaded in tension and shear.



Example 2.3

- **Given.** The web of a notched HEA 200 beam in steel grade S235 is connected with five M24 bolts (with $d_0 = 26$ mm) in two variations (a and b) for the positioning of the bolt group (fig. 2.37).
- **Question.** Determine for each variant the design value for the block tearing resistance $V_{\text{eff},2,\text{Rd}}$.
- **Answer.** In both variants the tearing pattern is un-symmetrical. In addition, the side of the tear pattern loaded in tension is eccentrically loaded. The design value of the block tearing resistance $V_{\text{eff},2,\text{Rd}}$ follows from equation (2.12). For variant a:

$$A_{\text{nt}} = t_w \left(e_2 - \frac{1}{2} d_0 \right) = 13 \cdot \left(45 - \frac{1}{2} \cdot 26 \right) = 416 \text{ mm}^2$$

$$A_{\text{nv}} = t_w \left(\ell_v + a_1 - n d_0 \right) = 13 \cdot (280 + 45 - 4,5 \cdot 26) = 2704 \text{ mm}^2$$

$$V_{\text{eff},2,\text{Rd}} = \frac{0,5 f_u A_{\text{nt}}}{\gamma_{\text{M2}}} + \frac{\frac{f_y A_{\text{nv}}}{\sqrt{3}}}{\gamma_{\text{M0}}} = \frac{0,5 \cdot 360 \cdot 416 \cdot 10^{-3}}{1,25} + \frac{\frac{235 \cdot 2704 \cdot 10^{-3}}{\sqrt{3}}}{1,0} = 427 \text{ kN}$$

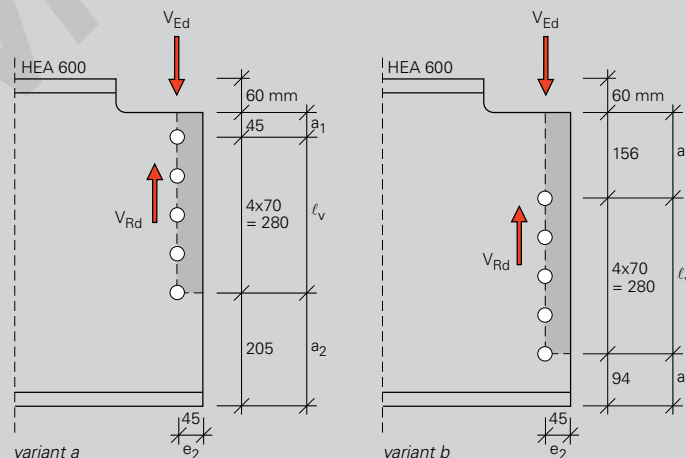
For variant b:

$$A_{\text{nt}} = t_w \left(e_2 - \frac{1}{2} d_0 \right) = 13 \cdot \left(45 - \frac{1}{2} \cdot 26 \right) = 416 \text{ mm}^2$$

$$A_{\text{nv}} = t_w \left(\ell_v + a_1 - n d_0 \right) = 13 \cdot (280 + 156 - 4,5 \cdot 26) = 4147 \text{ mm}^2$$

$$V_{\text{eff},2,\text{Rd}} = \frac{0,5 f_u A_{\text{nt}}}{\gamma_{\text{M2}}} + \frac{\frac{f_y A_{\text{nv}}}{\sqrt{3}}}{\gamma_{\text{M0}}} = \frac{0,5 \cdot 360 \cdot 416 \cdot 10^{-3}}{1,25} + \frac{\frac{235 \cdot 4147 \cdot 10^{-3}}{\sqrt{3}}}{1,0} = 623 \text{ kN}$$

This illustrates that the block tearing resistance strongly depends on the position of the bolts.



2.37 Notched beam connections with two variants of bolt placements.

2.5 Resistance of bolts

The resistance of individual, non-preloaded bolts depends on the way the bolt is loaded, namely:

- bolts loaded in shear (section 2.5.1);
- bolts loaded in tension (section 2.5.2);
- bolts loaded in shear and tension (section 2.5.3).

Design checks are given in table 3.2 of EN 1993-1-8. For bolts with a countersunk head some of the checks are modified (section 2.5.4).

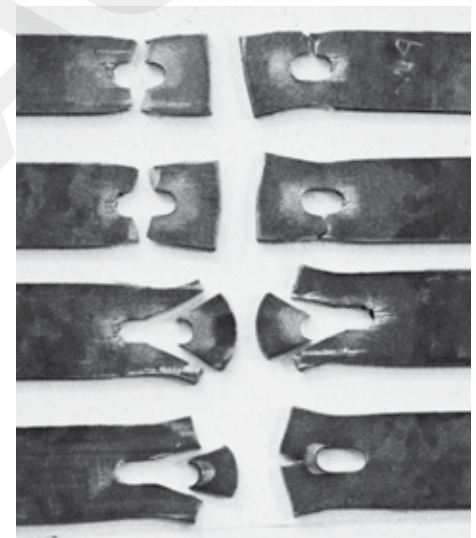
2.5.1 Bolts loaded in shear

A connection with non-preloaded bolts loaded in shear – see section 2.1.1 and figure 2.6 – will settle until the sides of the holes press against the bolt shafts. This settlement is reflected in the load/deformation behavior of the connection. The force distribution is schematically shown in figure 2.7a (category A). Different failure modes can determine the resistance of a connection, depending on the bolt diameter, the plate thickness, and the material characteristics of the bolts and plates (fig. 2.38 and 2.39).

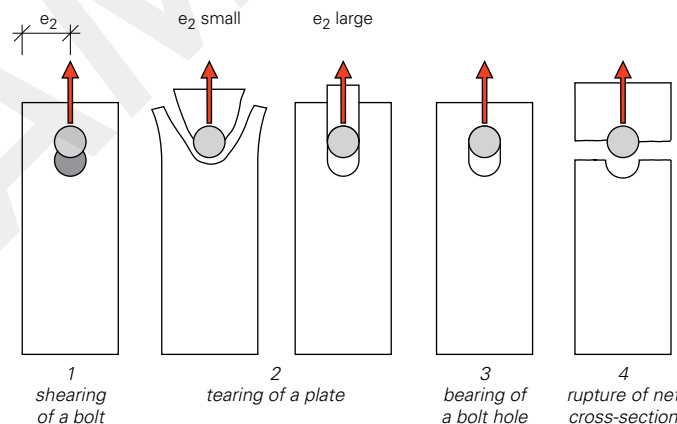
In principle, each possible failure mode must be checked individually; the failure mode with the lowest resistance is critical. However, this does not mean that it does not matter which failure mode governs. Shearing of a bolt (option 1 in fig. 2.39) occurs with relatively small deformation, while failure in bearing of a plate provides a larger deformation capacity. A design in which bearing is critical is preferable. EN 1993-1-8, cl. 3.6.1 combines tearing of a plate (option 2 in fig. 2.39) and bearing of a bolt hole (option 3 in fig. 2.39) in one failure mode. This combined failure mode is referred to as bearing resistance.

For non-preloaded bolts loaded in shear the following resistances are therefore important:

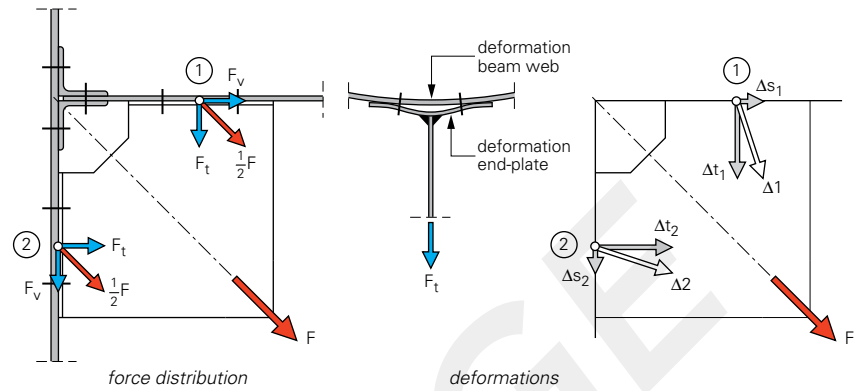
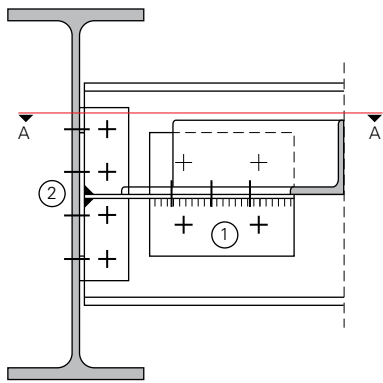
- shear resistance (shearing of a bolt);
- bearing resistance (tearing of a plate or bearing of a bolt hole).



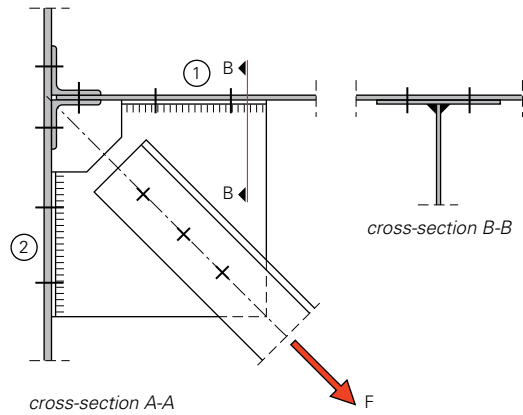
2.38 Different failure modes of failed test pieces.



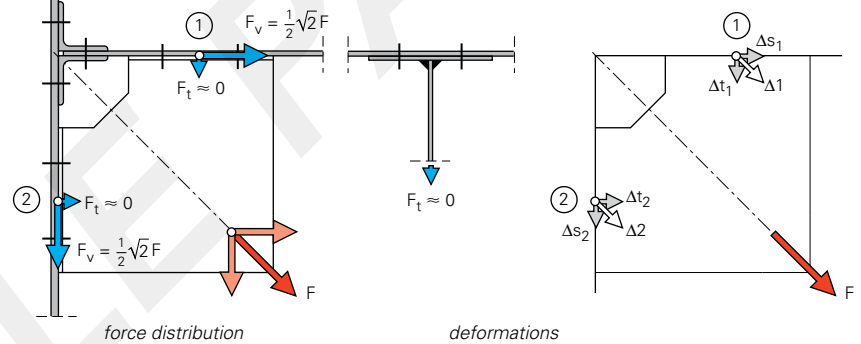
2.39 Possible failure modes of connections loaded in shear.



5.9 Incorrect assumption of the force distribution.



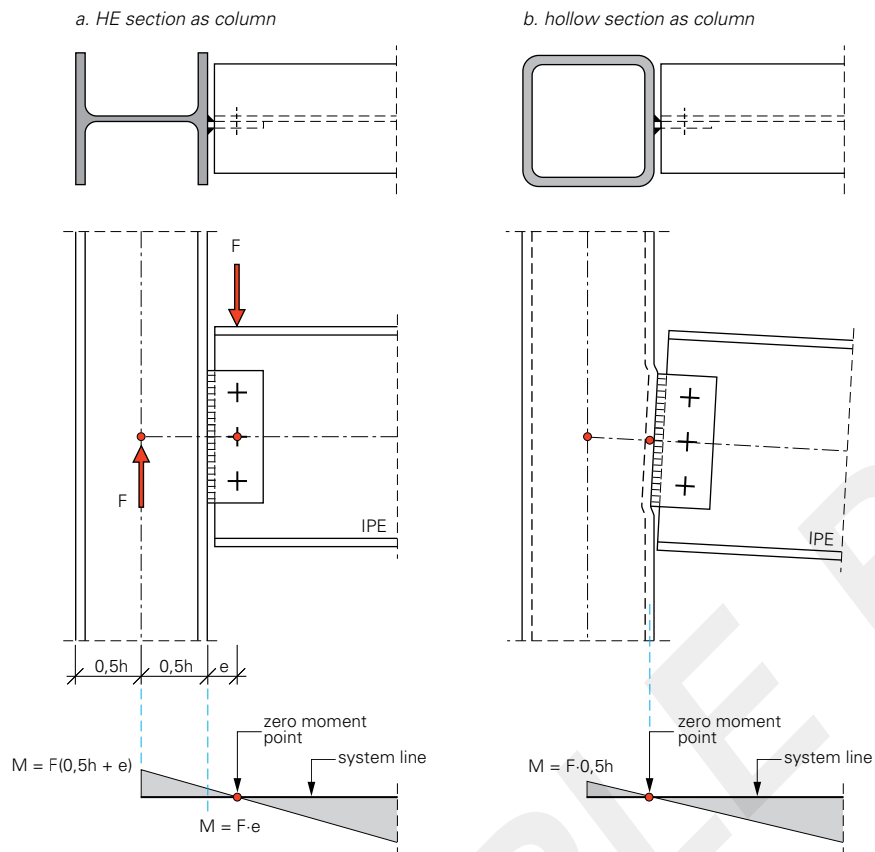
5.8 Connection of bracing in a floor structure.



5.10 Correct assumption of the force distribution.

Assume that the force is distributed simply in accordance with its direction (so evenly if the brace is at 45°) (fig. 5.9). However, for compatibility the deformation at (1) due to F_v must be equal to the deformation at (2) due to F_v , but given relative stiffnesses this would require F_v to be much larger than F_t . Therefore, the assumed evenly distributed forces cannot be correct. In figure 5.10 the correct force distribution is given: the force F causes shear in the bolt groups at (1) and (2).

Differences in stiffness can also influence the modelling of a structure. An example of this is a bolted connection between an IPE beam, which is nominally pinned to an HE column by means of a welded fin plate (fig. 5.11a). In this case, the deformation of the bolted connection will be larger than the deformation of the fin plate or the column web. A bending moment $M = F \cdot (e + 0,5h)$ and a compression force F act on the column, and a bending moment $M = F \cdot e$ and a shear force $V = F$ act on the welded connection. However if the column was a hollow section (fig. 5.11b), then the deformation of the hollow section wall would be critical. The zero moment point is now assumed to be at



5.11 Connection between a beam and an HE column (a) and a hollow section column (b).

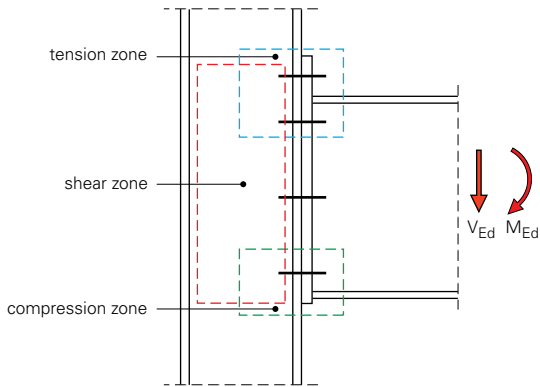
the location of the hollow section wall. In this case, a moment $M = F \cdot 0.5h$ and a compression force F act on the column, and only a shear force $V = F$ acts on the welded connection.

5.2.2 Centre of rotation

It has already been shown that in a bolted connection the distribution of forces between different bolts depends on the relative stiffnesses of the parts of the connection. The distance of a given bolt from the location of the centre of rotation is also of importance. A distinction can be made between free and forced rotation, depending on the detailing of the connection.

Free centre of rotation

Consider a connection with a random bolt pattern loaded by an eccentric force P_{Ed} (fig. 5.12a). In such a connection loaded in shear, it can be assumed that there are no stiffness differences given equal bolt and hole diameters, and therefore the centre of rotation coincides with the centre of



1.23 The three zones of load transfer in a bolted beam-to-column joint.

1.3 Resistance of the joint

For a beam-to-column joint, three separate zones are relevant to the transfer of forces: the tension zone, the shear zone, and the compression zone (fig. 1.23). The resistance of each zone is determined, and the lowest of the three dictates the joint resistance. In step 1, the components contributing to the strength in each zone are identified. The active components (indicated by a letter C) for this type of joint are listed in Table 1.24, which also contains the references to the relevant clauses in EN 1993-1-8.

The rules for the evaluation of the resistances of the active components in the three zones (step 2 in fig. 1.21) are discussed in sections 1.3.1 to 1.3.3 below. Once the resistance of each component has been determined, the resistance of each zone is determined as described in section 1.3.4, and from this the moment resistance $M_{j,Rd}$ (step 3 in fig. 1.21) is determined.

1.3.1 Tension resistance

The tension zone in an unstiffened bolted beam-to-column joint consists of the following six components:

- column flange in transverse bending (C4);
- end-plate in bending (C5);
- column web in tension (C3);
- beam web in tension (C8);
- bolts in tension (C10);
- welds (C19).

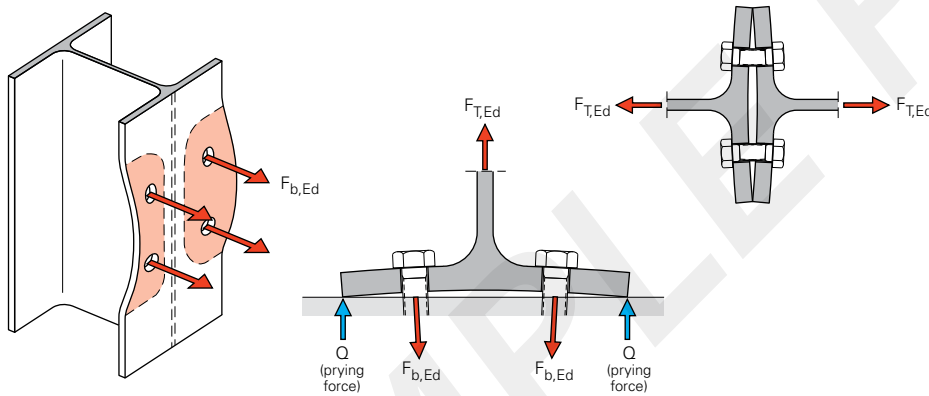
Column flange in transverse bending (component C4)

The bolted end-plate connection must transfer the entire tension force through bending of the column flange to the column web (fig. 1.25). To determine the design resistance of the column flange in bending, EN 1994-1-8, cl. 6.2.4, provides a model of an equivalent T-stub loaded in tension. Figure 1.26 shows the forces in such a T-stub for a single bolt row when the yield strength of the flange is critical. The resulting force in the bolts $F_{b,Ed}$ is in this case smaller than their tension resistance $F_{t,Rd}$. Apart from yielding of the flange, two other failure mechanisms can occur: brittle failure of the bolts with or without yielding of the flange.

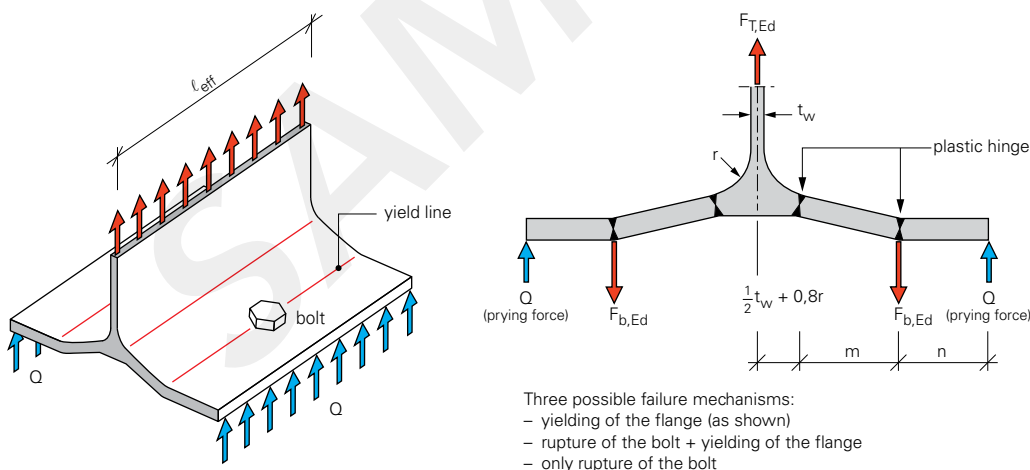
resistance	component	EN 1993-1-8
tension zone tension resistance $F_{t,Rd}$	C4 column flange in transverse bending	cl. 6.2.6.4
	C5 end-plate in bending	cl. 6.2.6.5
	C3 column web in transverse tension	cl. 6.2.6.3
	C8 beam web in tension	cl. 6.2.6.8
	C10 bolts in tension	cl. 6.2.6.4/5
	C19 welds	cl. 4 and cl. 6.2.3(5) ^[1]
shear zone shear resistance $V_{wp,Rd}$	C1 column web panel in shear	cl. 6.2.6.1
compression zone compression resistance $F_{c,Rd}$	C2 column web in transverse compression	cl. 6.2.6.2
	C7 beam flange and web in compression	cl. 6.2.6.7

1. In a beam-to-column joint where a plastic hinge must occur, in addition to the general weld assessment according to cl. 4, cl. 6.2.3(5) is also applicable.

1.24 Relevant components for the beam-to-column joint in figure 1.23.



1.25 Transfer of the tension force by bending of the column flange.



1.26 Model of an equivalent T-stub at ultimate limit state (yielding in the flange).

4

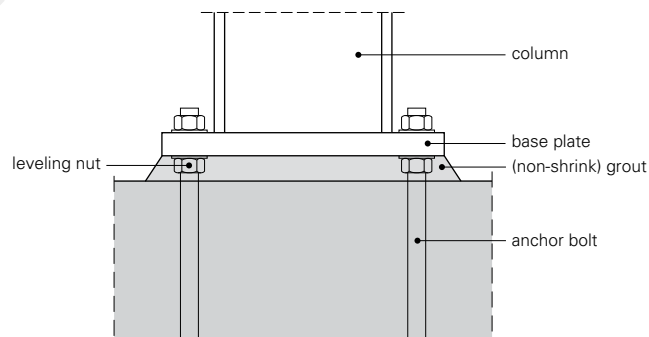
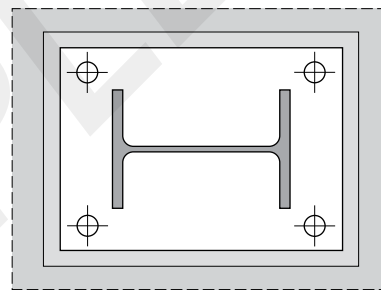
Column bases

A steel structure is almost always supported on a concrete foundation. To connect each steel column to the foundation, a base plate is usually welded to the column and then anchored to the concrete (fig. 4.1). The primary function of these joints is to transfer all column forces to the foundation. With nominally pinned joints, both a vertical compression force and a horizontal shear force usually need to be transferred. Moment-resistant joints (fixed column bases) must transfer a bending moment. If the column is part of a braced frame, tension forces may also occur in the joint. As well as transferring forces, the joint detailing must enable the column to be adjusted for line, level and plumb, and sometimes (temporarily) guarantee stability during erection.

This chapter first describes common types of column bases. Then the calculation of the resistance of centrally and eccentrically loaded base plates is discussed, as well as the determination of the joint stiffness. Finally, the fatigue of dynamically loaded base plates is briefly discussed, as well as some execution aspects.



4.1 Column base with an extending and a full-depth base plate.



4.2 Components of a column base with an extending base plate.

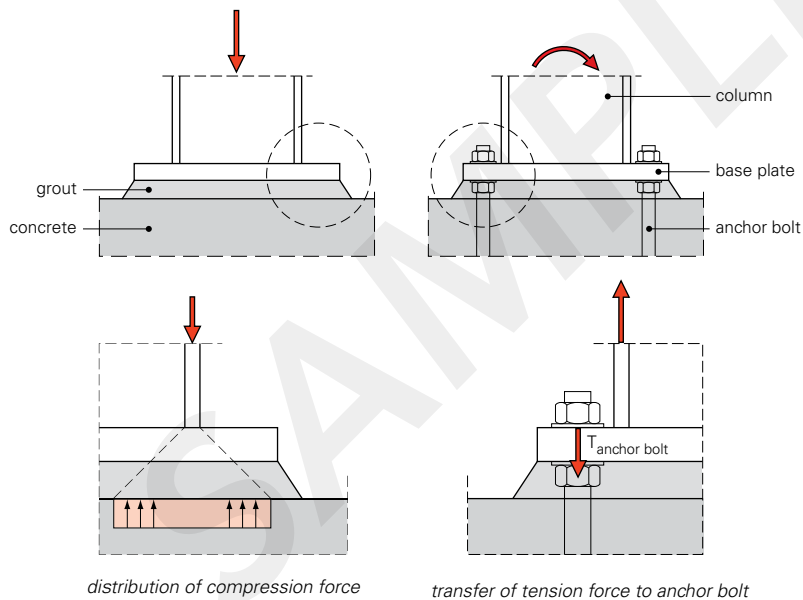
4.1 What are column bases?

The connection of a steel column to an underlying concrete foundation is normally achieved by welding a base plate to the column and connecting this base plate to the concrete via anchor bolts (fig. 4.2).

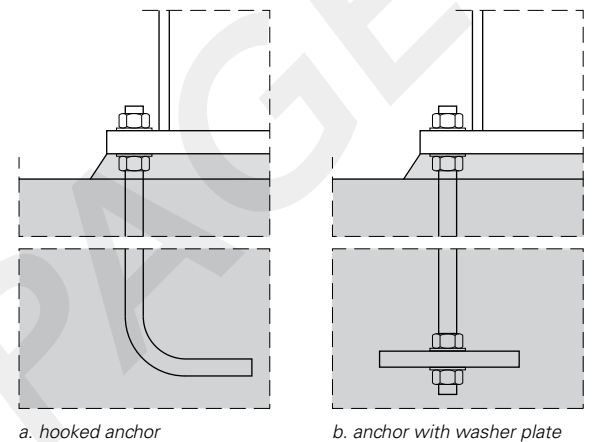
The base plate dimensions must be such that the compression force in the steel column is distributed over a sufficient area of the weaker concrete of the foundation or substructure. When the column loading is strongly eccentric – or fully in tension – the base plate must also be able to transfer the tension forces to the anchors by bending (fig. 4.3). The anchors must then transfer these tension forces to the concrete foundation.

EN 1993-1-8, cl. 6.2.6.12(4) notes that one of the following types of anchor should be used:

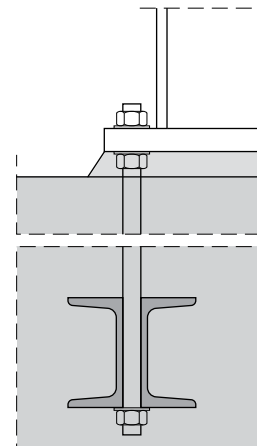
- cast in place anchor with a hook (fig. 4.4a);
- cast in place anchor with a washer plate (fig. 4.4b);
- other appropriate load-distributing member cast into the concrete, for example, a double U section for transferring high tension forces into the concrete (fig. 4.5). Such special details are not covered by this chapter;
- other fasteners which have been tested and approved, such as post-drilled anchor bolts. Such details are also not covered by this chapter.



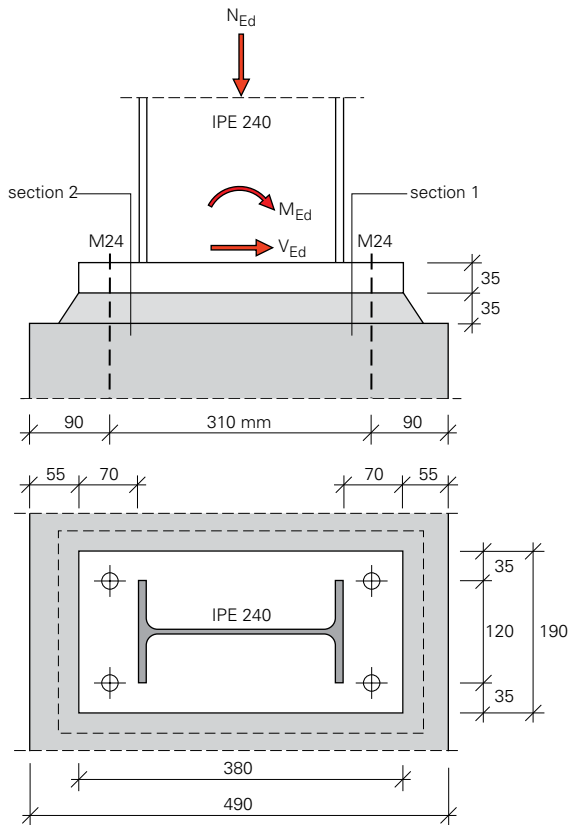
4.3 Functions of a base plate.



4.4 Hooked anchor and anchor with washer plate.



4.5 Heavier duty bespoke anchor.



4.32 Column base loaded in compression, bending and shear.

Example 4.2

• **Given.** A column base of an IPE 240 column in grade S235 steel, on a 490x800 mm concrete foundation beam in strength class C20/25, so $f_{ck} = 20 \text{ N/mm}^2$ (fig. 4.32). The column base is subject to a centric compression force $N_{Ed} = 85,5 \text{ kN}$, a moment $M_{Ed} = 75 \text{ kNm}$ and a shear force $V_{Ed} = 37,5 \text{ kN}$. The column is welded to a 35 mm thick base plate (190x380 mm) with double fillet welds ($a_w = 6 \text{ mm}$). The base plate is attached to the concrete with four M24 anchors (class 8.8; rolled thread), each with an 80x80x12 mm washer plate at a depth $h_{ef} = 400 \text{ mm}$. The grout layer under the base plate is of sand-cement mortar, with $f_{gk} = 20 \text{ N/mm}^2$ and a nominal thickness of 35 mm.

• **Question.** Assess the column base. The concrete foundation beam itself does not need to be assessed assuming that the axial forces from the column can be resisted by a pile foundation.

• **Answer.** First of all, the type of stress distribution is determined and the internal force distribution (compression force F_C and tension force T) is calculated. Secondly, the following resistances are calculated and assessed:

- compression resistance;
- tension resistance;
- shear resistance.

Type of stress distribution

The load eccentricity is $e = M_{Ed}/N_{Ed} = 75 \cdot 10^3 / 85,5 = 877 \text{ mm}$ and the centre-to-centre distance between column flanges equals $z_1 = h_a - t_f = 240 - 9,8 = 230 \text{ mm}$. Therefore: $e = 877 \text{ mm} > 0,5z_1 = 0,5 \cdot 230 = 115 \text{ mm}$, so calculation model type II applies, with one compression stress zone and a tension force in the two left hand side anchors (see fig. 4.18).

Internal force distribution

The compression force F_C and the tension force T in both anchors together follow from equation (4.12):

$$z_{II} = 0,5z_1 + z_T = 0,5 \cdot 230 + 0,5 \cdot 310 = 270 \text{ mm}$$

$$T = \frac{M_{Ed} - 0,5z_1 N_{Ed}}{z_{II}} = \frac{75 - 0,5 \cdot 230 \cdot 10^{-3} \cdot 85,5}{270 \cdot 10^{-3}} = 241 \text{ kN}$$

$$F_C = N_{Ed} + T = 85,5 + 241 = 327 \text{ kN}$$

Compression resistance

The calculation and assessment of the design compression resistance involves the following steps:

- calculate the bearing strength f_{jd} ;
- calculate the additional bearing width c ;
- calculate the dimensions of the stress footprint;
- calculate and assess the design compression resistance $N_{fl,Rd}$;
- assess whether splitting reinforcement is required.

Bearing strength

The bearing strength f_{jd} is determined (conservatively) using equation (4.8). As a starting point the length of the cantilever is used for the additional bearing width: $c = 70$ mm. The distance from the edge of the base plate to the edge of the concrete foundation beam is $c_d = 55$ mm:

$$b_1 = b_{\text{eff}} = t_f + 2c = 9,8 + 2 \cdot 70 = 150 \text{ mm}$$

$$d_1 = \ell_{\text{eff}} = b_a + 2c = 120 + 2 \cdot 70 = 260 \text{ mm} > b_p = 190 \text{ mm} \Rightarrow d_1 = 190 \text{ mm}$$

$$A_{c0} = b_1 d_1 = 150 \cdot 190 = 28500 \text{ mm}^2$$

$$A_{c1} = b_2 d_2 = (b_1 + 2c_d) 3d_1 = (150 + 2 \cdot 55) \cdot 3 \cdot 190 = 148200 \text{ mm}^2$$

$$k_d = \sqrt{\frac{A_{c1}}{A_{c0}}} = \sqrt{\frac{148200}{28500}} = 2,3 \leq 3,0 \text{ (critical)}$$

$$f_{cd} = \frac{\alpha_{cc} f_{ck}}{\gamma_c} = \frac{1,0 \cdot 20}{1,5} = 13,3 \text{ N/mm}^2$$

$$f_{jd} = \beta_j k_d f_{cd} = \frac{2}{3} \cdot 2,3 \cdot 13,3 = 20,4 \text{ N/mm}^2$$

The grout material meets the requirement in equation (4.6): $f_{gk} = 20 \text{ N/mm}^2 \geq 0,2f_{ck} = 0,2 \cdot 20 = 4 \text{ N/mm}^2$. The grout thickness also meets the requirement: $t_g = 35 \text{ mm} \leq 0,2b_p = 0,2 \cdot 190 = 38 \text{ mm}$.

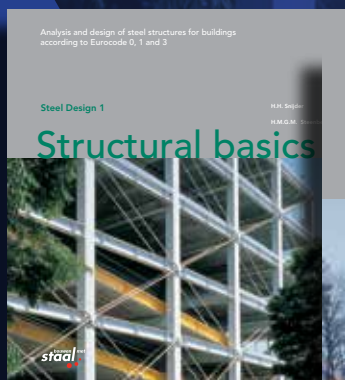
Additional bearing width

The additional bearing width c follows from equation (4.3):

$$c = t_p \sqrt{\frac{f_y}{3f_{jd}}} = 35 \cdot \sqrt{\frac{235}{3 \cdot 20,4}} = 68,6 \text{ mm}$$

NA

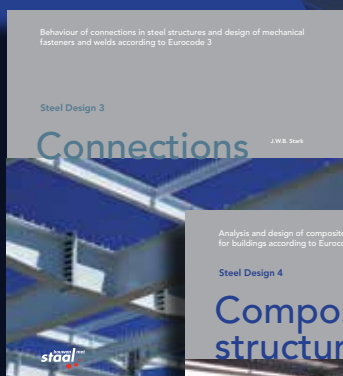
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