Analysis and design of steel structures for buildings according to Eurocode 0, 1 and 3

Steel Design 1

H.H. Snijder

H.M.G.M. Steenbergen

Structural basics

bouwen met staal

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Colophon

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Content

7 Resistance of cross-sections 7-2

Structural basics

2

Actions and deformations

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2 Actions and deformations

2.1 Structures are subjected to several types of actions, like wind.

Structures are subjected to several types of actions (fig. 2.1), which each can vary in magnitude and can occur in combination with each other. EN 1991 defines the different types of actions, which should be taken into account in the design of a structure. When combining these actions, partial factors are taken into account. Combinations of actions are used when checking if the requirements are met concerning safety (load bearing capacity) and serviceability (deformations) of the structure. This chapter discusses the following topics concerning actions and deformations:
– structural requirements and relevant concepts;
– structural safety;
– permanent loads;
– serviceability criteria;
– actions according to EN 1991.

Finally, three examples are given to illustrate the actions on a floor of a house, a free-standing canopy and an office building.

2.1 Structural requirements and relevant concepts

The Eurocodes provide principles and application rules which need to be satisfied. Principles comprise general statements and definitions for which there is no alternative, and requirements and analytical models for which no alternative is permitted unless specifically stated. EN 1990 allows the use of alternative design rules different from the application rules mentioned in the Eurocodes provided that it is shown that the alternative rules comply with the relevant principles and are at least equivalent with regard to structural safety, serviceability and durability, which would be expected when using the Eurocodes. Also properties of products mentioned in quality declarations issued and approved by acknowledged certification institutes may be used, like properties mentioned in a European Assessment Document based on a European Technical Assessment (ETA) by a recognized Technical Assessment Body.
For the assessment of the load bearing structure of a building, the requirements concerning safety (load bearing capacity) and serviceability (deformations) are especially important. The former means that the ultimate limit state shall not be exceeded; the latter that the serviceability limit state should not be violated. Exceeding an ultimate limit state occurs when there is insufficient resistance (strength) or insufficient stability (of a part) of the structure (fig. 2.2).

This section briefly discusses the concepts of ‘actions’, ‘combination of actions’ and ‘limit state’.

2.1.1 Actions

The term 'action' includes all things which could cause stresses or deformations in a structure. Examples are forces, moments, imposed deformations and temperature differences.
Design values of actions should be based on their characteristic values, or on other representative values. A design value F_d is determined as follows:

$$F_d = \gamma_f F_{rep} \quad \text{with} \quad F_{rep} = \psi F_k \tag{2.1}$$

Where:

- γ_f partial factor for actions which takes account of the possibility of unfavourable deviations from the representative values;
- F_{rep} representative value of the action;
- ψ factor: $\psi = 1{,}0$ or $\psi = \psi_0$, ψ_1 or ψ_2 (see section 2.4);
- F_k characteristic value of the action.

The effects of an action E are defined as the consequences of the action which occur at specific locations, cross-sections or elements of a structure (for example internal forces – such as moments, shear forces and normal forces – or for example stresses or strains), or on the structure as a whole (for example deflection, displacement, misalignment or rotation). The representative value of the effect of an action E_{rep} is determined from the representative value of the action and the geometrical data. The design value of the effect of the action E_d is obtained by multiplying by a partial factor γ_F:

$$E_d = \gamma_F E_{rep} \tag{2.2}$$

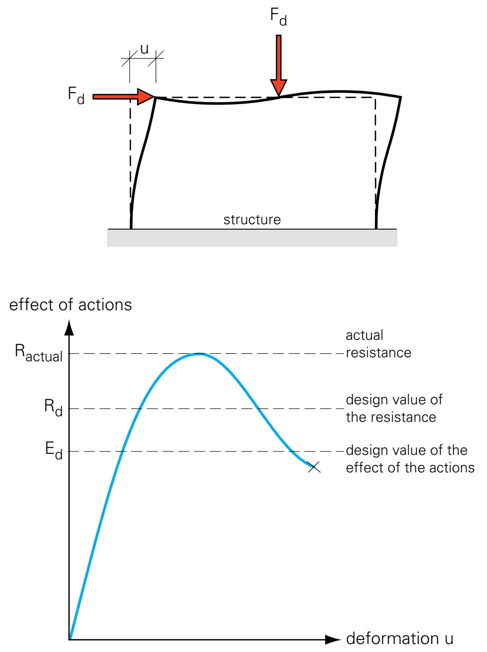
Where:

- γ_F partial factor for actions: for permanent load γ_G, for variable action γ_Q and for accidental actions γ_A (see table 2.6);
- E_{rep} representative value of the effect of an action.

The influences of a number of uncertainties and of the variations in the basic actions are incorporated in the partial factor for actions $\gamma_F = \gamma_f \cdot \gamma_{Sd}$. These are related to the action itself and to the structure. These influences are:

- the possibility of unfavourable deviations of the action value from the representative value (γ_f);
- uncertainties in modelling the effect of the action – for example the presence of unfavourable deviations such as out-of-plumpness and eccentricities – which mean that the schematization (design model) does not correspond with reality (γ_{Sd}).

In principle, a statistical approach applies where the action is defined as a characteristic value with a specific probability of exceedance. EN 1991 provides the characteristic values of actions



2.2 A structure should be designed such that the ultimate limit state is not exceeded.

buckling curve	elastic analysis	plastic analysis
	e_0/L	e_0/L
a_0	1/350	1/300
a	1/300	1/250
b	1/250	1/200
c	1/200	1/150
d	1/150	1/100

4.8 Design values of local initial bow imperfections.

For frames in buildings, the effect of imperfections may be neglected if the horizontal load on a storey – for example wind load – is relatively large. In such cases the effect of imperfections is negligibly small compared to the effect of the larger horizontal load on the specific storey. This is the case if:

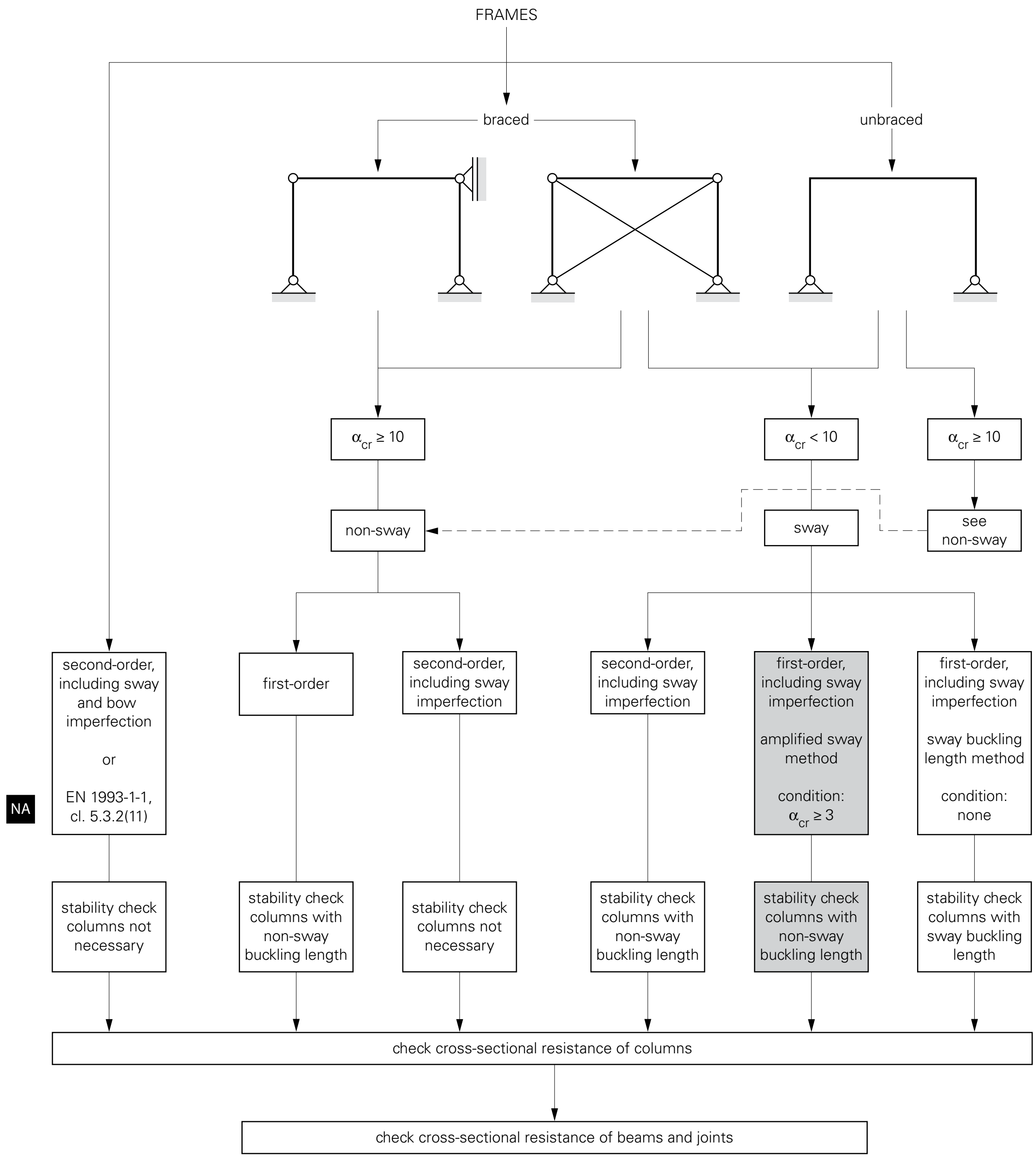
$$H_{Ed} \geq 0{,}15 V_{Ed} \quad (4.7)$$

Where:
H_{Ed} design value of the total horizontal load transferred by the story;
V_{Ed} design value of the total vertical load on a frame transferred by a story.

The imperfections at a member level are local initial bow imperfections e_0, as shown in figure
NA 4.6b. Table 5.1 in EN 1993-1-1, cl. 5.3.2(3) presents the initial bow imperfection e_0 for flexural buckling in relation to the member length L and provides, per buckling curve and per analysis method, a value for e_0/L (table 4.8).
According to EN 1993-1-1, cl. 5.3.2(7), it is also allowed for this case to apply equivalent horizontal loads instead of local initial bow imperfections (see fig. 4.7b).

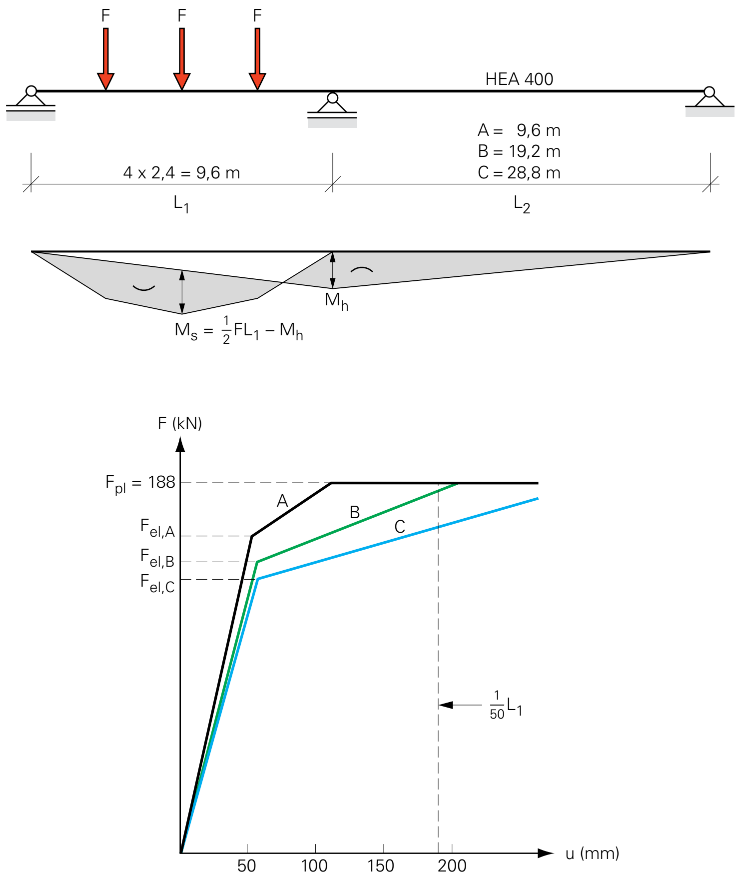
4.1.4 Analysis methods for frames

For an analysis, the frame is first defined as a mechanics model which represents the shape of the structure, the support conditions, the cross-sections and the joints. Then the frame must be classified as either sway or non-sway. Next, the imperfections should be taken into account and a suitable analysis method for determining the force distribution and deformations should be chosen. After this, verifications against the various design rules presented in the code are carried out.
EN 1993-1-1 provides several analysis methods for frames depending on the classification into braced/unbraced and sway/non-sway frames (table 4.5). The methods and options for an elastic global analysis are summarized in figure 4.9.

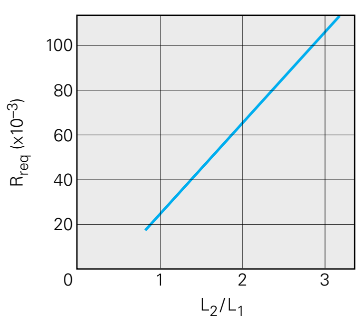


4.9 Analysis methods for frames (grey boxed part: for buildings only).

In this section, first the methods that may be used for the global analysis to determine the force distribution and deformations, using a full and an approximate second-order analysis, are discussed. Next, elastic and plastic global analyses are discussed. Finally, consideration is given to how the design rules relate to the analysis methods.



5.16 Influence on the deflection of the stiffness ratio between the loaded and unloaded parts of a continuous beam HEA400 of steel grade S235.



5.17 Required relative rotation capacity of the beam of figure 5.16.

5.1.3 Stiffness ratios

Additional considerations of deformation compatibility are not necessary in order to determine the plastic resistance. The resistance follows directly from the equilibrium of the statically determinate failure mechanism. When there are extreme stiffness ratios between the elements – (parts of) beams, columns and joints – of the structure, it is possible that extremely large deformations are required for the theoretical resistance to be reached. This can mean that the predicted resistance has little practical meaning, as illustrated by the example below.

Consider the beam on three supports (HEA 400 section in S235) shown in figure 5.16. Assume that the left span, with a length of L_1 = 9,6 m, is loaded. The right span is not loaded and its length is varied, for example: $L_2 = L_1$, $L_2 = 2L_1$ and $L_2 = 3L_1$. The degree of rotational restraint provided at the middle support for the loaded left span depends on the length of the right span. Figure 5.16 shows the moment diagram and the relationship between the load F and the deflection u.

The first plastic hinge occurs for all three cases in the left span under the central point load. As the length of the right span increases, the stiffness of the beam decreases significantly after the occurrence of the first plastic hinge. For $L_2 = 3L_1$ (branch C), the slope of the load-deflection diagram after reaching F_{el} is particularly shallow, so that the theoretical resistance F_{pl} is then only reached after very large deflections. In that case, the theoretical resistance does not have practical meaning as such large deformations would not be acceptable.

To prevent this type of situation and to obtain a usable definition of the resistance, it could be agreed that the value of the resistance should be determined for a plastic deformation u_{pl} which is smaller than an agreed limit, for example $L_1/50$. When the deflection goes beyond this limiting value there is no assumed increase in resistance. Although EN 1993-1-1 does not contain such a limiting value, it is nevertheless recommended to limit the plastic deformation.

In figure 5.17, the relative required rotation capacity R_{req} of the first plastic hinge is plotted against the ratio of the length of the two spans. This relative required rotation capacity increases with the length L_2 of the unloaded span. This is an additional reason for limiting the plastic deflection. Such a situation can occur for beams which are semi-rigidly connected to a column, for example by a relatively flexible end plate joint.

5.1.4 Initial sway imperfection

Imperfections should always be taken into account in the analysis of a frame (fig. 5.18). A distinction can be made between:
- global initial sway imperfections of the frame as a whole;
- local initial bow imperfections of the columns.

The influence of local initial bow imperfections of the columns is not discussed further in this chapter, but rather reference should be made to the stability of individual members. A global initial sway imperfection ϕ can have a relatively large influence on the first-order plastic resistance F_{pl} of a frame. This is especially the case when there are no (or limited) horizontal loads on the frame and the vertical loads on the columns are large relative to the vertical loads on the beams (fig. 5.19). The resistance can be determined as follows (fig. 5.20):

beam mechanism

$$F\theta \frac{1}{2}L = M_{pl,Rd}4\theta$$

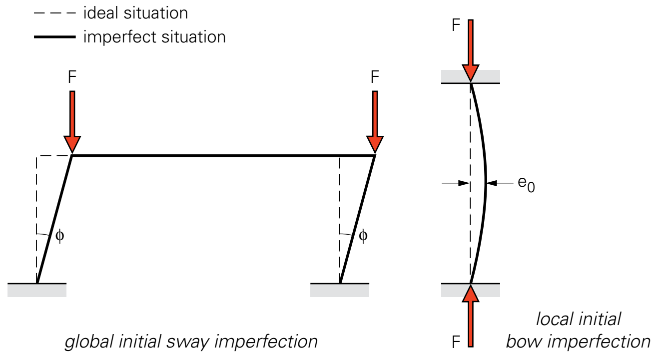
$$F = F_{pl,beam} = \frac{8M_{pl,Rd}}{L}$$

sway mechanism: the first-order failure mechanism follows from the equilibrium of the frame including global initial sway imperfection ϕ:

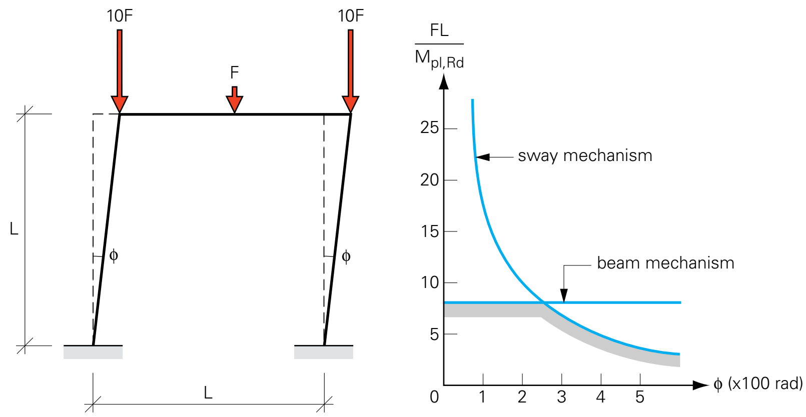
$$21F\theta\phi L = M_{pl,Rd}4\theta$$

$$F = F_{pl,sway} = \frac{4M_{pl,Rd}}{21\phi L}$$

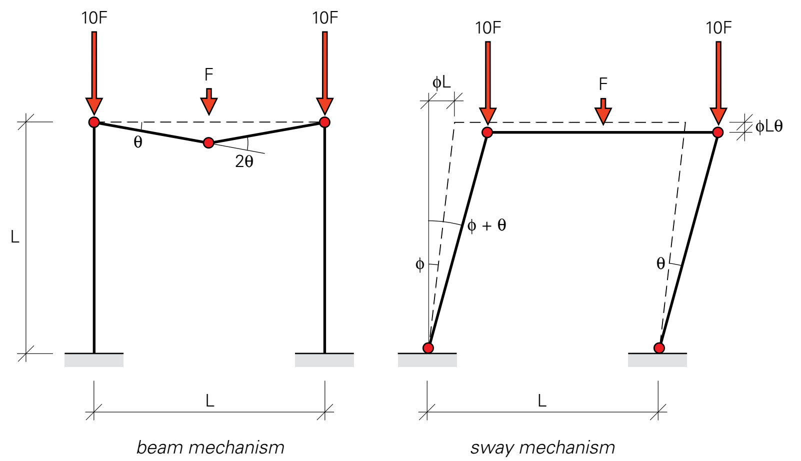
In this example, the sway mechanism is critical for $\phi > 0{,}025$ despite the absence of a horizontal load. Due to its possibly large influence on F_{pl}, it is always required to take the global initial sway imperfection into account. This can be done either by considering a frame which is out-of-plumb, to include the global initial sway imperfection directly in the geometry, or by considering equivalent horizontal loads per floor in the analysis. The imperfections to be taken into account are given in EN 1993-1-1 cl. 5.3.



5.18 Global initial sway and local initial bow imperfections.



5.19 Influence of the global initial sway imperfection f on the frame resistance.



5.20 The two possible failure mechanisms.

Example 7.5

- *Given*. A simply supported HEA 280 beam in steel grade S235 with a span of L = 8 m (fig. 7.9). The beam is subject to a uniformly distributed load q_{Ed} = 8,7 kN/m and a point load F_{Ed} = 69,6 kN at mid span.
- *Question*. Check the resistance of the beam at mid span.
- *Answer*. The design shear force V_{Ed} at the support and the maximum bending moment M_{Ed} at mid span are:

$$V_{Ed} = \frac{1}{2}q_{Ed}L + \frac{1}{2}F_{Ed} = \frac{1}{2}\cdot 8,7\cdot 8 + \frac{1}{2}\cdot 69,6 = 69,6\ \text{kN}$$

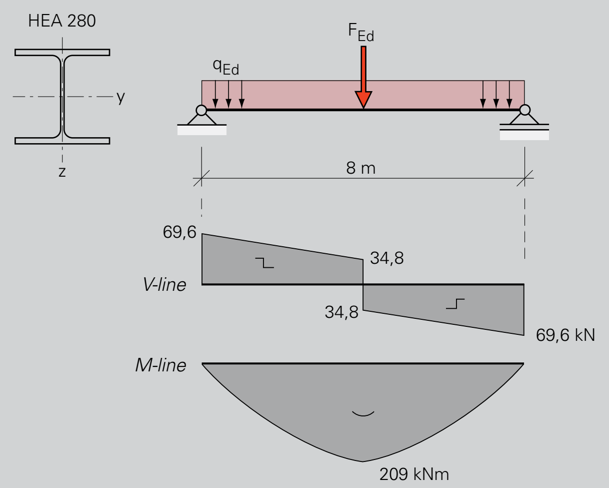
$$M_{Ed} = \frac{1}{8}q_{Ed}L^2 + \frac{1}{4}F_{Ed}L = \frac{1}{8}\cdot 8,7\cdot 8^2 + \frac{1}{4}\cdot 69,6\cdot 8 = 209\ \text{kNm}$$

The moment and shear force distributions are shown in figure 7.10. At mid span there is a shear force (V_{Ed} = 34,8 kN) and a moment (M_{Ed} = 209 kNm) acting in combination. For $V_{Ed} \leq 0{,}5V_{pl,Rd}$ the influence of the shear force on the moment resistance may be neglected. The shear area is A_v = 3174 mm^2 and the shear resistance $V_{pl.Rd}$ = 431 kN (see example 7.4). Therefore:

$$V_{Ed} = 34{,}8\ \text{kN} \leq 0{,}5V_{pl,Rd} = 0{,}5\cdot 431 = 216\ \text{kN}$$

This means that the influence of shear on the bending moment resistance may be neglected. Equation (7.13) should be used for the assessment of the bending moment. For the cross-section classification and moment resistance see example 7.3. The check is then:

$$\frac{M_{Ed}}{M_{c,Rd}} = \frac{209}{261} = 0{,}80 \leq 1{,}0\ \text{(OK)}$$



7.9 Simply supported beam with uniformly distributed load and a concentrated load at mid span.

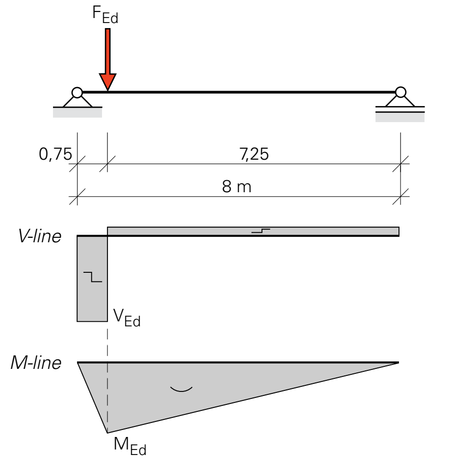
7.10 Shear and bending moment distribution of the beam shown in figure 7.9.

Example 7.6

- *Given*. A simply supported HEA 280 beam in steel grade S235 with a span of L = 8 m. The beam is subject to a concentrated load F_{Ed} = 323 kN at a distance L_1 = 0,75 m from the support (fig. 7.11).
- *Question*. Check the resistance of the cross-section at the location of the concentrated load.
- *Answer*. The design values of shear force and bending moment under the concentrated load are:

$$V_{Ed} = \frac{L - L_1}{L} F_{Ed} = \frac{8 - 0,75}{8} \cdot 323 = 293 \text{ kN}$$

$$M_{Ed} = L_1 V_{Ed} = 0,75 \cdot 293 = 220 \text{ kN}$$



7.11 Simply supported beam with a concentrated load close to a support, with corresponding shear and bending moment distributions.

At the location of the concentrated load, a shear force (V_{Ed} = 293 kN) and bending moment (M_{Ed} = 220 kNm) are present acting in combination.
For $V_{Ed} \leq 0,5V_{pl,Rd}$, the influence of the shear force on the bending moment resistance may be neglected. In example 7.4, $V_{pl,Rd}$ = 431 kN was determined, so:

$$V_{Ed} = 293 \text{ kN} > 0,5V_{pl,Rd} = 0,5 \cdot 431 = 216 \text{ kN}$$

This means that the influence of shear on the bending moment resistance may not be neglected. For an I-section with identical flanges in bending about the major axis, the reduced design moment resistance allowing for shear may be determined according to equation (7.40) with $A_w = h_w t_w$ (see fig. 7.7b), and with the factor ρ according to equation (7.37):

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1 \right)^2 = \left(\frac{2 \cdot 293}{431} - 1 \right)^2 = 0,13$$

$$A_w = h_w t_w = \left(h - 2t_f\right) t_w = \left(270 - 2 \cdot 13\right) \cdot 8 = 1952 \text{ mm}^2$$

NA

$$M_{y,V,Rd} = \frac{\left(W_{pl,y} - \frac{\rho A_w^2}{4t_w} \right) f_y}{\gamma_{M0}} = \frac{\left(1112 \cdot 10^3 - \frac{0,13 \cdot 1952^2}{4 \cdot 8} \right) \cdot 235 \cdot 10^{-6}}{1,0}$$

$$= 258 \text{ kNm} \leq M_{y,c,Rd} = 261 \text{ kNm}$$

According to equation (7.41) the check is:

$$\frac{M_{Ed}}{M_{y,V,Rd}} = \frac{220}{258} = 0,85 \leq 1,0 \text{ (OK)}$$

Steel Design 1

Structural basics

This textbook covers the design and analysis of steel structures for buildings according to EN 1990 (Eurocode 0), EN 1991 (Eurocode 1) and EN 1993 (Eurocode 3).

- Chapter 1 describes the theory and background of EN 1990 in terms of structural safety, reliability and the design values of resistances and actions.
- Chapter 2 deals with actions and deformations described in EN 1991. The permanent loads and variable actions and in particular the imposed loads and the snow loads and wind actions are discussed. This chapter also contains three worked examples to determine the actions on a floor in a residential house, the actions on a free-standing platform canopy at a station and the wind actions on the façades of an office building.
- Chapter 3 is about modelling, discussing the schematization of the structural system, the joints and the material properties as well as the cross-section properties.
- Chapter 4 deals with the classification of frames and the various analysis methods for unbraced and braced frames.
- Chapter 5 then goes deeper into these analysis methods to determine the force distribution and deformations.
- Chapter 6 deals with the assessment by code-checking of (parts of) the steel structure with EN 1993-1-1 and EN 1993-1-8. At a basic level, the assessment of the resistance of cross-sections, the stability of members under axial forces and the resistance of bolted and welded connections are explaned.
- Chapter 7 discusses in an extensive way the assessment by code-checking of the resistance of cross-sections, both for single and combined internal forces. The principles of the assessment of the resistance of cross-sections according to elastic and plastic theory are also discussed.

Structural basics is effective as a textbook for students and as a reference guide to the Eurocodes 0, 1 en 3 for practising structural engineers.

The authors – prof.ir. H.H. Snijder and ir. H.M.G.M. Steenbergen – are experienced tutors in the field of (steel) structures.

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- Henri Steenbergen is a senior researcher at The Netherlands Organisation for applied scientific research (TNO), specialised in (structural) safety and the behavior and analysis of (steel) structures. He is often involved in special projects in which the structure must be assesed based on advanced calculation techniques or on the backgrounds of (steel) standards.

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