STEEL BUILDINGS IN EUROPE

Multi-Storey Steel Buildings
Part 4: Detailed Design

FOREWORD

This publication is part four of the design guide, *Multi-Storey Steel Buildings*.

The 10 parts in the *Multi-Storey Steel Buildings* guide are:

Part 1: Architect's guide
Part 2: Concept design
Part 3: Actions
Part 4: Detailed design
Part 5: Joint design
Part 6: Fire Engineering
Part 7: Model construction specification
Part 8: Design software – section capacity
Part 9: Design software – simple connections
Part 10: Software specification for composite beams.

Multi-Storey Steel Buildings is one of two design guides. The second design guide is *Single-Storey Steel Buildings.*

The two design guides have been produced in the framework of the European project "Facilitating the market development for sections in industrial halls and low rise buildings (SECHALO) RFS2-CT-2008-0030".

The design guides have been prepared under the direction of Arcelor Mittal, Peiner Träger and Corus. The technical content has been prepared by CTICM and SCI, collaborating as the Steel Alliance.

Contents

SUMMARY

This document is the fourth part of a publication covering all aspects of medium rise multi-storey building design. The guide focuses on the detailed design of buildings that use steel bracing or concrete cores to resist horizontal actions and provide horizontal stability.

The objective of this document is to introduce the basic concepts of multi-storey form of construction, commonly referred to as ‘simple construction’ and to provide guidance on practical aspects of building design.

It provides detailed guidance on how to design for stability, and goes on to give advice on the global analysis of multi-storey buildings.

It also covers the serviceability and ultimate limit state requirements of EN 1993 and EN 1994 and provides guidance on design for robustness to meet the requirements of EN 1991-1-7.

It includes six worked examples, covering common elements in the design of multi-storey buildings.

1 INTRODUCTION

1.1 General

In this publication, medium rise steel frames are defined as frames where neither resistance to horizontal loads, nor achieving sufficient sway stability has significant impact on either the plan arrangement of the floors or the overall structural form. This limit is normally regarded as twelve storeys.

Low rise buildings (of two or three storeys) are only subject to modest horizontal forces and may readily be conceived with robust bracing systems such that second order effects are minimised, to the extent that sway stability effects need not be considered explicitly in design. The bracing may be provided either by triangulated bracing or by reinforced concrete core(s); the floors act as diaphragms to tie all columns into the bracing or cores.

1.2 Scope of this document

This document guides the designer through all the steps involved in the detailed design of braced multi-storey frames to EN 1993[1] and EN 1994[2].

It focuses on the application of 'simple construction' as the way of achieving the most economic form of construction. Coincidently, and very conveniently, these approaches also are the simplest to use in the design office, thereby minimising design office costs.

The guide addresses:

- The basic concept of simple construction
- Guidance on the global analysis of frames for simple construction
- Design checks at the Serviceability Limit State (SLS)
- Design checks for the Ultimate Limit State (ULS): floor systems, columns, vertical and horizontal bracing
- Checks to ensure the structure has sufficient robustness to resist both specified and unspecified accidental loads.

2 BASIC CONCEPTS

2.1 Introduction

EN 1993-1-1[1] provides a very flexible, comprehensive framework for the global analysis and design of a wide range of steel frames.

This section introduces the basic concepts that underpin the design approaches for economic low and medium-rise multi-storey frames.

2.2 Simple construction

As discussed in *Multi-storey steel buildings. Part 2: Concept design*[3], the greatest economy for low and medium rise braced multi-storey frames will be achieved by the use of 'simple construction'. The analysis assumes nominally pinned connections between beams and columns; resistance to horizontal forces is provided by bracing systems or concrete cores. Consequently, the beams are designed as simply supported and the columns are designed only for any moments arising from nominal eccentricities of connections of the beams to the columns (in conjunction with the axial forces). As a further consequence, it is not necessary to consider pattern loading to derive design forces in the columns.

This design approach is accommodated by the EN 1993[1] 'simple' joint model, in which the joint may be assumed not to transmit bending moments. This approach may be used if the joint is classified as 'nominally pinned' according to EN 1993-1-8, § 5.2.2. This classification may be based on previous satisfactory performance in similar cases. The joint configurations discussed in Section 3.3 assume a pinned connection and that the beam reactions are applied eccentrically to the columns. The widespread and successful use of these types of connection in many parts of Europe provide the evidence of satisfactory performance required by EN 1993-1-8, § 5.2.2.

For braced frames designed in accordance with EN 1993-1-1[1], the global analysis model may therefore assume pinned connections between the columns and the beams, provided that the columns are designed for any bending moments due to eccentric reactions from the beams (see Section 3.3).

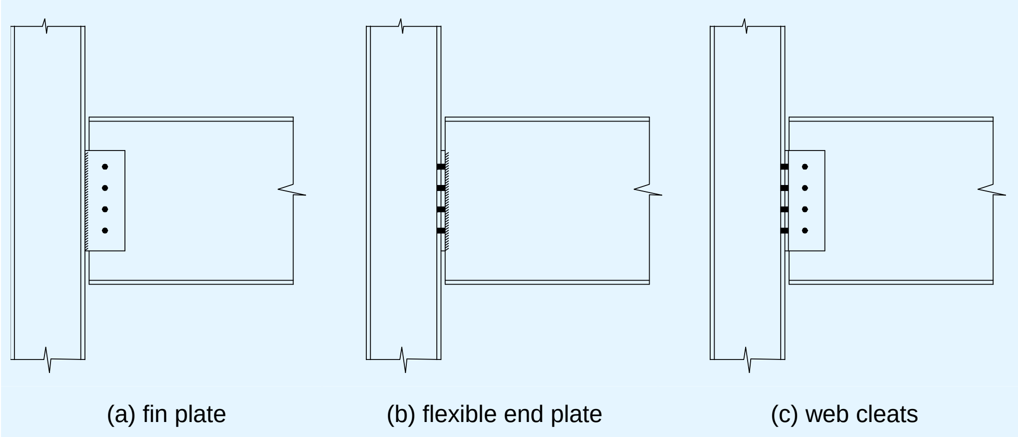


Figure 2.1 Typical 'simple' beam to column connections

For such simple frames, it is almost always economic to use:

- HE sections for columns
- IPE sections for beams
- Flats, angles or hollow sections for diagonal bracing members.

Figure 2.1 shows typical beam to column connections.

2.3 Sway and non-sway frames

2.3.1 Definitions

A braced frame has sufficient structural components to transmit horizontal forces directly to the foundations. These components provide stability to the frame. They may be one or more concrete cores, which will usually contain the vertical services, lifts and stairs. Alternatively, they may be complete systems of triangulated steel members in vertical planes (acting in conjunction with floor diaphragms or horizontal bracing).

In a braced frame, the beams are designed as simply supported. The columns carry axial loads and (generally) minimal moments. The beam to column connections are designed as nominally pinned, and hence not attracting any moment; sufficient rotation capacity must be provided.

An unbraced frame is any frame which does not have either a concrete core or a complete system of vertical triangulation. At least some beam to column connections must be moment resisting in order to transmit horizontal forces to the foundations and to provide frame stability.

It should be noted that horizontal structure and associated behaviour needs to be considered separately in two, usually orthogonal, directions. Thus a frame may be:

- Braced in both directions
- Braced in one direction and unbraced in the other
- Unbraced in both directions

A ‘sway-sensitive frame’ is a frame where horizontal flexibility is such that there needs to be some allowance for the effects of deformed geometry.

A ‘sway-insensitive frame’ is a frame with sufficient horizontal stiffness that second order effects may be ignored.

It should be noted that horizontal stiffness needs to be considered separately in two, usually orthogonal, directions. Thus, a frame may be:

- A sway-sensitive frame in both directions
- A sway-sensitive frame in one direction and a sway-insensitive frame in the other
- A sway-insensitive frame in both directions.

2.3.2 Distinction between sway/non-sway and unbraced/braced concepts

Both sway-sensitivity and braced/unbraced concepts relate to horizontal structure. However, they are essentially different.

Sway-sensitivity definitions entirely relate to horizontal stiffness and behaviour.

Braced/unbraced are descriptions of structural arrangement.

It follows that, in each of the two orthogonal planes, a frame may be:

- Braced and sway-insensitive
- Braced and sway-sensitive
- Unbraced and sway-sensitive
- Unbraced but sway-insensitive (unusual, but possible).

2.4 Second order effects

2.4.1 Basic principles

The sensitivity of any frame to second order effects may be illustrated simply by considering one ‘bay’ of a multi-storey building in simple construction (i.e. with pinned connections between beams and columns); the bay is restrained laterally by a spring representing the bracing system. First and second order displacements are illustrated in Figure 2.2.

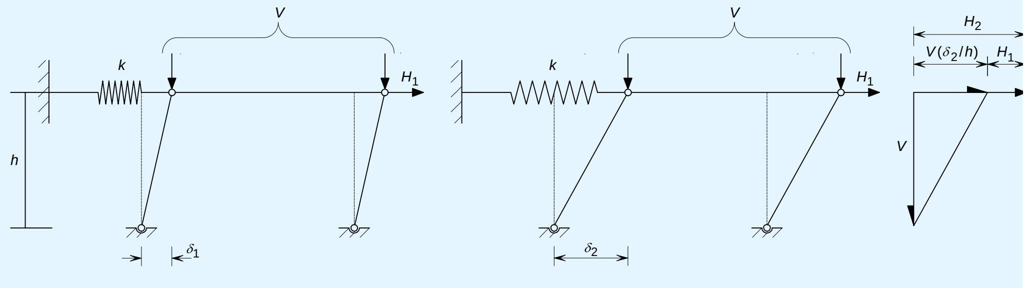


Figure 2.2 First and second order effects in a pinned braced frame

The equilibrium expression for the second order condition may be rearranged as:

$$H_2 = H_1 \left(\frac{1}{1 - V/kh} \right)$$

Thus, it can be seen that, if the stiffness k is large, there is very little amplification of the applied horizontal force and consideration of first order effects only would be adequate. On the other hand, if the external horizontal force, H_1, is kept constant while the value of total vertical force V tends toward a critical value V_{cr} (= kh), then displacements and forces in the restraint tend toward infinity. The ratio V_{cr}/V, which may be expressed as a parameter α_{cr} is thus an indication of the second order amplification of displacements and forces in the bracing system due to second order effects. The amplifier is given by:

$$\left(\frac{1}{1 - 1/\alpha_{cr}} \right)$$

EN 1993-1-1[1] presents both general rules and specific rules for buildings. In order to cover all cases, § 5.2.1 of that code considers the applied loading system, F_{Ed}, comprising both horizontal forces H_{Ed} and vertical forces V_{Ed}. The magnitudes of these forces are compared to the elastic critical buckling load for the frame, F_{cr}. The measure of frame stability, α_{cr} is defined as $\dfrac{F_{cr}}{F_{Ed}}$.

Although F_{cr} may be determined by software or from stability functions, the Eurocode provides a simple approach to calculate α_{cr} directly in § 5.2.1(4)B:

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}} \right) \left(\frac{h}{\delta_{H,ED}} \right)$$

where:

α_{cr} is the factor by which the design loading would have to be increased to cause elastic instability in a global mode

H_{Ed} is the design value of the horizontal reaction at the bottom of the storey to the horizontal loads and the equivalent horizontal forces

V_{Ed} is the total design vertical load on the structure on the bottom of the storey

$\delta_{H,Ed}$ is the horizontal displacement at the top of the storey, relative to the bottom of the storey, under the horizontal loads (both externally applied and equivalent horizontal forces)

h is the storey height.

2.4.2 Allowance for second order effects

As discussed in Section 2.3.1, horizontal flexibility influences overall structural stability and the significance of second order effects on overall design.

As described in Section 2.4.1, EN 1993-1-1 § 5.2.1 introduces the concept of α_{cr} as the basic measure of horizontal flexibility and its influence on structural stability.

Depending on the value of α_c, three alternative design situations are possible.

$\alpha_{cr} > 10$

Where horizontal stability is provided by a concrete core, or by robust bracing, calculations will generally demonstrate that $\alpha_{cr} > 10$ for all combinations of actions. EN 1993-1-1, § 5.2.1(3) permits the use of first order analysis for such frames.

When $\alpha_{cr} > 10$, second order effects are considered small enough to be ignored.

It may be convenient for certain low rise frames to ensure that $\alpha_{cr} > 10$, by providing bracing of sufficient strength and stiffness. This is discussed in Section 2.6. For medium rise structures, this simple approach will usually lead to heavy triangulated bracing with large and expensive connections.

$3{,}0 < \alpha_{cr} < 10$

For buildings between three and ten storeys, bracing designed for strength will generally lead to $3{,}0 < \alpha_{cr} < 10$. (If α_{cr} should fall below 3,0 it is usually practical to increase bracing sizes to satisfy this lower limit).

For $\alpha_{cr} > 3{,}0$ EN 1993-1-1, § 5.2.2(6)B permits the use of first order analysis provided that all storeys a similar:

- distribution of vertical loads and
- distribution of horizontal loads and
- distribution of frame stiffness with respect to the applied storey shear forces.

To allow for second order effects, all relevant action effects are amplified by the factor

$$\frac{1}{1-\dfrac{1}{\alpha_{cr}}}$$

Relevant action effects are:

- Externally applied horizontal loads, e.g. wind, H_{Ed}
- Equivalent horizontal forces (EHF) used to allow for frame imperfections, $V_{Ed}\phi$
- Other possible sway effects. (These are rare in low rise multi-storey frames but could occur, for example where the building is on a sloping site with differing levels of foundations. In such circumstances, axial shortening of the different lengths of columns will cause overall sway.)

α_{cr} < 3.0

For $\alpha_{cr} < 3.0$, EN 1993-1-1, § 5.2.2 requires such structures to be analysed by second order analysis. This approach is not recommended for low or medium rise buildings. Second order analysis may lead to marginal economies in the mass of bracing systems but this advantage is more than offset by the increased design costs and the difficulty of optimising such structures. Such flexible structures are anyway likely to fail horizontal serviceability criteria.

2.5 General design procedure

Unless the simple approach described in Section 2.6 for low rise buildings is adopted, the general design process is as follows:

1. Determine the ULS vertical actions.
2. Calculate the equivalent horizontal forces (EHF) to allow for imperfections (see Section 2.7.1 of this guide).
3. Determine the ULS horizontal loads.
4. Determine the total horizontal loads (from 2 and 3 above).
5. Choose bracing configuration and choose bracing members, based on the total horizontal loads.

   (Note that the wind forces and EHF are usually distributed to individual braced bays by simple load allocation techniques, thus avoiding the need for a three-dimensional analysis).
6. Carry out first order analysis of the braced frames to determine both the forces in the bracing system and the sway stiffness of the frames.

   (This two dimensional analysis of each vertical bracing system is usually carried out by computer to provide ready access to displacements).
7. For each floor of each braced bay, determine the ‘local’ α_{cr} from:

   $$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\frac{h}{\delta_{H,Ed}}$$
8. Determine the governing α_{cr} as the lowest value obtained from the analysis above.
9. If $\alpha_{cr} > 10$, second order effects are small enough to be ignored.
   If $3{,}0 < \alpha_{cr} < 10$, calculate the amplification factor and increase all relevant action effects (the bracing may need to be re-designed).

If $\alpha_{cr} < 3,0$ the recommended approach is to increase the stiffness of the structure.

2.6 Design of steel bracing systems to achieve $\alpha_{cr} \geq 10$ for all combinations of actions

2.6.1 Introduction

Vertical bracing is designed to resist wind load plus equivalent horizontal forces given by EN 1993-1-1, § 5.3. First order frame analysis can be used for braced frames, provided that the vertical bracing provides sufficient stiffness. For first order analysis to be applicable, EN 1993-1-1, § 5.2.1 requires that $\alpha_{cr} \geq 10$ for the whole frame and therefore for each storey of a multi-storey building.

Simple guidance is given below for the selection of bracing members so that sufficient stiffness is provided for such analysis to be valid. This allows the designer to avoid either the complexities of second order analysis, or of allowing for second order effects by amplification of first order effects. The method also permits the design of the frame to be undertaken without any recourse to computer analysis (such analysis is normally necessary in order to determine horizontal displacements and hence, α_{cr}).

The parametric study that led to these design recommendations is presented in Access Steel document SN028a-EN-EU[4].

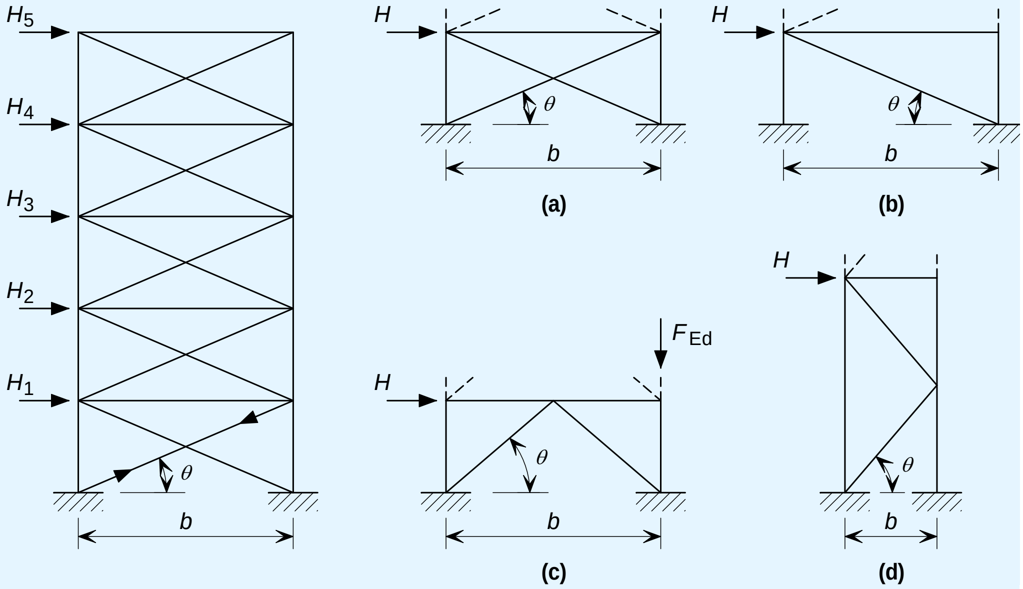
The bracing arrangements considered by this study are presented in Figure 2.3.

2.6.2 Scope

The design procedure presented below was derived for buildings with the following limitations:

- Height not exceeding 30 m
- Angle of bracing members between 15° and 50° to the horizontal
- The bracing arrangements are as shown in Figure 2.3

Note that the procedure does not depend on the steel grade.



At each floor level, $H_i = 0{,}025 \times V_{Ed,i}$ where $V_{Ed,i}$ is the total design load applied at that floor level

(a) cross bracing, only diagonal in tension participating
(b) diagonal bracing
(c) horizontal K bracing
(d) vertical K bracing

Figure 2.3 Practical alternative arrangements for multi-storey bracing:

2.6.3 Design procedure

Select one of the bracing arrangements shown in Figure 2.3.

Verify that, in the columns and beams of the system to be braced, the axial stresses calculated on the gross cross-section due to resistance of the horizontally applied loads of 2,5% of vertical applied loads alone do not exceed 30 N/mm^2. (This is to limit the elongations of the bracing and shortenings in the columns.) If the stresses are higher in the columns, either larger sections must be chosen, or the spacing of the columns '*b*' in Figure 2.3, must be increased (but not exceeding 12 m). If the stresses in the beams are larger, either a larger section must be chosen or the bracing arrangement must be changed.

Size the bracing by conventional design methods, to resist horizontal applied loads of 2,5% of vertical applied loads, ensuring that axial stresses on the gross cross-section of the bracing do not exceed the values given in Table 2.1. For intermediate floors, either the stress limits in Table 2.1 for the top floor should be used, or a higher stress may be found by linear interpolation between the stress limits according to the height of the bottom of the storey considered.

If the externally applied horizontal load, plus the equivalent horizontal forces from imperfections, plus any other sway effects calculated by first-order analysis, exceed 2,5% of the vertical loads, check the resistance of the bracing to these loads. The stress limitations in Table 2.1 should not be applied when checking this load combination.

Table 2.1 Limiting stress on the gross cross-section of the bracing members

Angle of bracing to the horizontal θ (degrees)	Stress limit on the gross cross-section of the bracing member due to horizontal forces equal to 0.025V		
	Top storey of 30 m building	Top storey of 20 m building	Bottom storey of building
$15 \leq \theta < 20$	65 N/mm^2	80 N/mm^2	100 N/mm^2
$20 \leq \theta < 30$	70 N/mm^2	95 N/mm^2	135 N/mm^2
$30 \leq \theta < 40$	55* N/mm^2	110 N/mm^2	195 N/mm^2
$40 \leq \theta < 50$	75 N/mm^2	130 N/mm^2	225 N/mm^2

* This value is lower than the rest due to the way in which the forces are distributed

Note: The maximum stresses in Table 2.1 are limited in application to a building of a maximum height of 30 m, storey height ≥ 3 m, with 5 m ≤ b ≤ 12 m. The maximum permissible axial stress on the gross cross-section of the columns and beams (from horizontal loads of 2,5% of vertical loads) is 30 N/mm^2.

2.7 The effects of imperfections

Four types of imperfections influence the behaviour and design of multi-storey frames and their components. The references listed below relate to EN 1993-1-1.

- Overall sway imperfections (§ 5.3.2 (1) to (4))
- Sway imperfections over a storey (§ 5.3.2 (5))
- Imperfections at splices (§ 5.3.3 (4))
- Individual bow imperfections of members (§ 5.3.4).

EN 1993-1-1 provides comprehensive guidance on the treatment of all four types of imperfection.

2.7.1 Overall sway imperfections

The global sway imperfections to be considered are shown in EN 1993-1-1 Figure 5.2, reproduced below as Figure 2.4.

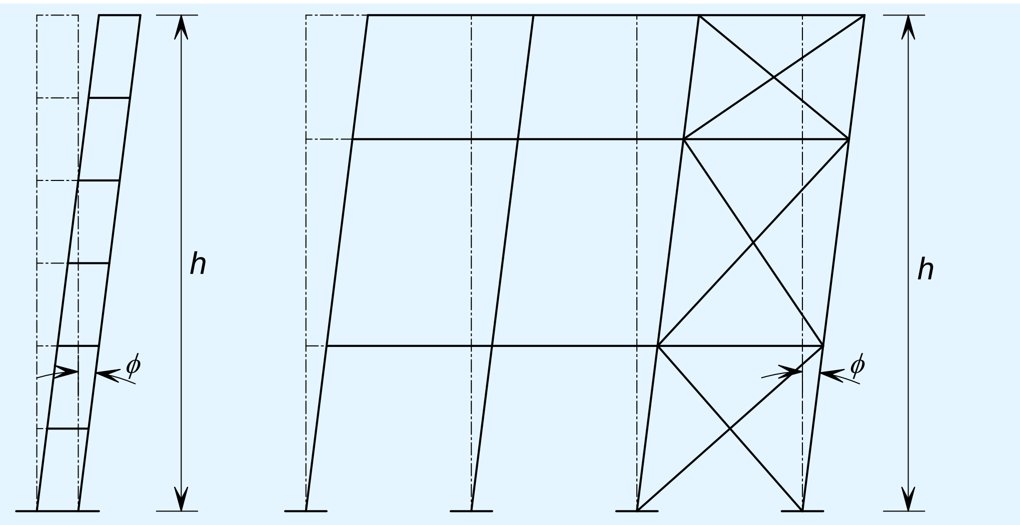


Figure 2.4 Equivalent sway imperfections (taken from EN 1993-1-1 Figure 5.2)

The basic imperfection is an out-of-verticality ϕ of 1/200. This allowance is greater than normally specified erection tolerances, because it allows both for actual values exceeding specified limits and for effects such as lack of fit and residual stresses.

The design allowance in EN 1993-1-1, § 5.3.2 is given by:

$$\phi = \phi_0 \; \alpha_h \; \alpha_m = \frac{1}{200} \alpha_h \; \alpha_m$$

where:

α_h is a reduction factor for the overall height and

α_m is a reduction factor for the number of columns restrained by the bracing system. (For detailed definition, see EN 1993-1-1, § 5.3.2(3)).

For simplicity, the value of ϕ may conservatively be taken as 1/200, irrespective of the height and number of columns.

Where, for all the storeys, the horizontal force exceeds 15% of the total vertical force, sway imperfections may be neglected (because they have little influence on sway deformation and amplification factor for such robust structures).

2.7.2 Use of equivalent horizontal forces

EN 1993-1-1, § 5.3.2(7) states that vertical sway imperfections may be replaced by systems of equivalent horizontal forces, introduced for each column. It is much easier to use equivalent horizontal forces than to introduce the geometric imperfection into the model. This is because:

- The imperfection must be tried in each direction to find the greater effect and it is easier to apply loads than modify geometry.
- Applying forces overcomes the problems of the changes in length that would occur when considering the columns of buildings in which the column bases are at different levels.

According to EN 1993-1-1, § 5.3.2(7) the equivalent horizontal forces have the design value of ϕN_{Ed} at the top and bottom of each column, where N_{Ed} is the force in each column; the forces at each end are in opposite directions. For design of the frame, it is much easier to consider the net equivalent force at each floor level. Thus an equivalent horizontal force equal to ϕ times the total vertical design force applied at that floor level should be applied at each floor and roof level.

2.7.3 Sway imperfections over a storey

The configuration of imperfections to be considered over a storey assumes a change in direction of the column at that level, as shown in Figure 2.5. The inclined columns produce a horizontal force (the horizontal component of the inclined force). This horizontal force must be transferred to the stability system (the bracing or concrete core(s)) via the floor diaphragm or via horizontal bracing designed for that purpose. Usually it is sufficient to transfer these forces via the floor diaphragm.

Figure 2.5 shows two cases, both of which give rise to a horizontal shear force of ϕN_{Ed}. Note that in this case, the value of ϕ is calculated using a value of α_h that is appropriate to the height of only a single storey and that, since the value of N_{Ed} is different above and below the floor, the larger value (i.e. that for the lower storey) should be used.

2.7.4 Imperfections at splices

EN 1993-1-1, § 5.3.3 states that imperfections in the bracing system should also be considered. Whereas most of the clause is applicable to bracing systems that restrain members in compression, such as chords of trusses, the guidance on forces at splices in § 5.3.3(4) should be followed for multi-storey columns.

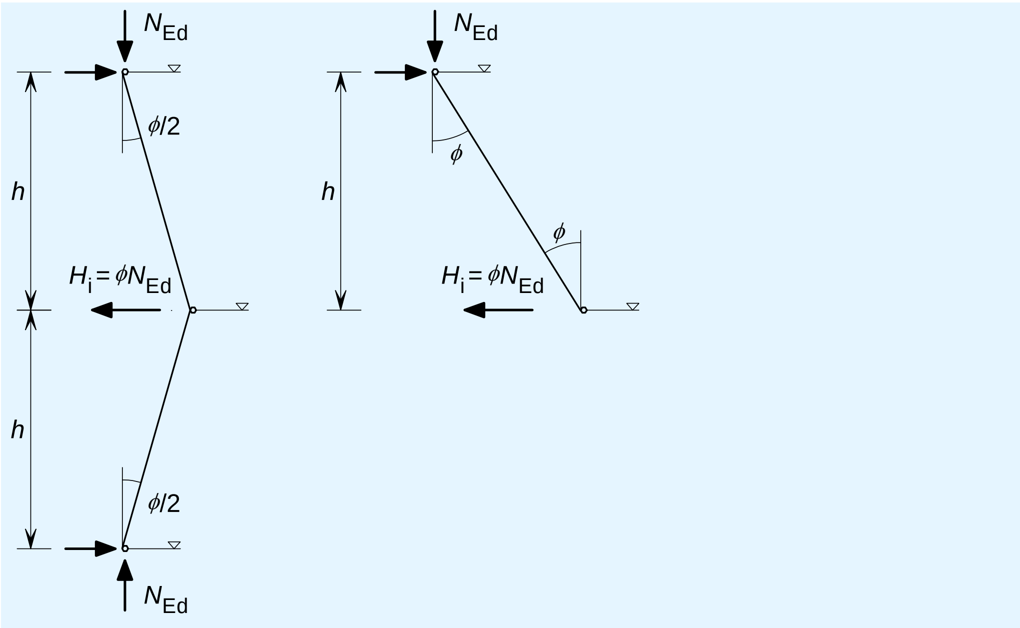
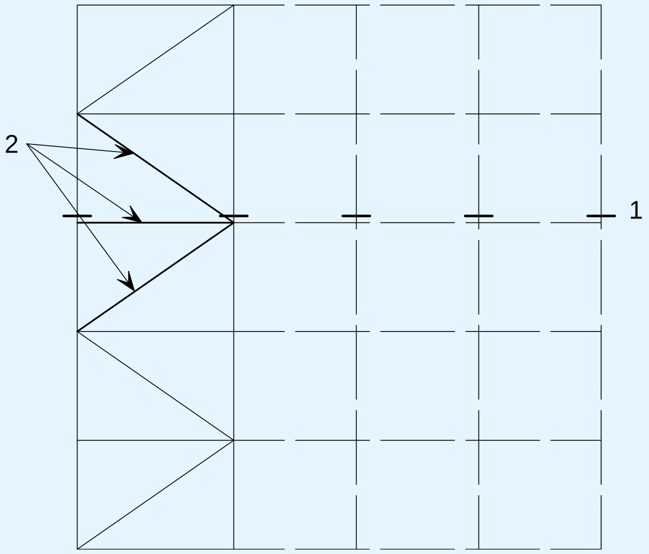


Figure 2.5 Configuration of sway imperfection ϕ for horizontal forces on floor diaphragm (taken from EN 1993-1-1, Figure 5.3)

The lateral force at a splice should be taken as $\alpha_m N_{Ed}/100$, and this must be resisted by the local bracing members in addition to the forces from externally applied actions such as wind load but excluding the equivalent horizontal forces. The force to be carried locally is the summation from all the splices at that level, distributed amongst the bracing systems. If many heavily-loaded columns are spliced at the same level, the force could be significant. Assuming that a splice is nominally at a floor level, only the bracing members at that floor and between the floor above and below need to be verified for this additional force. This is shown in Figure 2.6.

This additional force should not be used in the design of the overall bracing systems and is not taken to the foundations, unless the splice is at the first storey. When designing the bracing system, only one imperfection needs to be considered at a time. When checking the bracing for the additional forces due to imperfections at splices, the equivalent horizontal forces should not be applied to the bracing system.



1 Level of splice
2 Local members to be verified for additional force arising from (in this case) 5 splices

Figure 2.6 Bracing members to be verified at splice levels

As the force may be in either direction, it is advised that the simplest approach is to divide the force into components (in the case above, into the two diagonal members) and verify each member for the additional force. Note that the values of the imperfection forces and the forces in the members due to wind load vary, depending on the combination of actions being considered.

2.7.5 Member bow imperfections

In a braced frame with simple connections, no allowance is needed in the global analysis for bow imperfections in members because they do not influence the global behaviour. The effects of local bow imperfections in members are taken into account in the design of both compression members and unrestrained beams through the use of appropriate buckling curves, as described in Section 6 of EN 1993-1-1.

2.7.6 Design recommendations for imperfections

Based on the background studies presented in Access Steel document SN047a[4], it is possible to make the following simple, safe recommendations for design. (More direct application of EN 1993-1-1 could reduce the design imperfection forces by 50% in some circumstances, but the forces are normally small).

1. Apply equivalent horizontal forces of 1/200 of vertical forces at floor and roof levels. Storey shears at any level in the building will be 1/200 of the total forces above (the summation of the EHF above that level).

   These forces should be considered in all relevant horizontal directions but need only be considered in one direction at a time.

   (In accordance with EN 1993-1-1, § 5.3.2(10), the possible torsional effects on a structure caused by anti-symmetric sways at the two opposite faces should also be considered. This effect is only significant in a building of very low torsional stiffness on plan, a situation which is unlikely to occur in practice).

2. Verify that all columns are tied into all attached beams by connections with a minimum resistance of 1,0% of the column axial force, i.e. that the tying resistance of the beam to column connection is at least 0,01 × $N_{\text{Ed,column}}$.

3. Verify that all the equivalent horizontal forces in each column can be transferred into the relevant bracing system. Diaphragm action in the floor slab may be mobilised to satisfy this condition.

   In accordance with EN 1993-1-1, § 5.3.3(1), a reduction factor

   $$\alpha_{\text{m}} = \sqrt{0,5\left(1 + \frac{1}{m}\right)}$$

   may be applied, where m is the number of columns to be restrained.

2.8 Design summary

- 'Simple' design and construction provides the most economical approach for low and medium rise frames.
- System(s) of triangulated bracing or concrete core(s) provide resistance to horizontal forces and overall frame stability.
- 'Simple' connections have been proven by experience to provide sufficient strength, shear and rotation capacity to satisfy the assumptions of this method.
- Beams are designed to span between grid lines.
- Columns are designed for axial loading only, with no account of pattern loading and only nominal moments. The value of the nominal moments will be based on the equilibrium model adopted for the connections, as described in Section 3.3.
- Concrete cores may generally be assumed to provide sufficient stiffness for all potential second order effects to be ignored.
- For low-rise (2 or 3 storey) braced frames, designing the bracing for horizontal actions of 2,5% of vertical actions in accordance with section 2.6 will provide sufficient horizontal stiffness that all potential second order effects may be ignored.
- Frames of intermediate stiffness, where $3 < \alpha_{\text{cr}} < 10$, may be analysed by first order analysis, providing all relevant actions are amplified by the factor

  $$\frac{1}{1 - \frac{1}{\alpha_{\text{cr}}}}$$

- Frames with $\alpha_{\text{cr}} < 3,0$ should be avoided.
- The effects of sway imperfections and imperfections at splices may simply be addressed by applying equivalent horizontal forces of 1/200 of vertical forces, in accordance with Section 2.7.6 of this publication.

3 PRACTICAL GLOBAL ANALYSIS FOR 'SIMPLE CONSTRUCTION'

3.1 Introduction

This Section provides guidance on the global analysis of a low or medium rise building, taking appropriate account of the specific aspects of frame behaviour addressed in Section 2. The Section addresses both persistent and transient design situations. Design for accidental situations is addressed in Section 6.

3.2 Actions and their combinations

Buildings have to be designed for the combinations of actions set out in EN 1990[5], § 6.4.3.2; this aspect is discussed in more detail in *Multi-storey steel buildings. Part 3: Actions*[6]

For the ultimate limit state, the basic combination of actions is given in expression (6.10) as:

$$\sum_{j\geq 1}\gamma_{\mathrm{G},j}G_{\mathrm{k},j} + \gamma_{\mathrm{p}}P + \gamma_{\mathrm{Q},1}Q_{\mathrm{k},1} + \sum_{i>1}\gamma_{\mathrm{Q},i}\psi_{0,i}Q_{\mathrm{k},i} \qquad 6.10$$

This combination includes the permanent actions $G_{\mathrm{k},j}$, the pre-stressing action P (not normally applicable in multi-storey steel building frames), the leading variable action $Q_{k,1}$ and the various accompanying variable actions $Q_{\mathrm{k,i}}$. Partial factors, γ, are applied to the characteristic value of each action and additionally a factor ψ_0 is applied to each accompanying action.

Alternatively, for the STR and GEO Limit States (see EN 1990-1-1 § 6.4.1), EN 1990[5] permits the use of the least favourable of the combinations of actions given in expressions (6.10a) and (6.10b) for the ultimate limit state**.**

$$\sum_{j\geq 1}\gamma_{\mathrm{G},j}G_{\mathrm{k},j} + \gamma_{\mathrm{p}}P + \gamma_{\mathrm{Q},1}\psi_{0,1}Q_{\mathrm{k},1} + \sum_{i>1}\gamma_{\mathrm{Q},i}\psi_{0,i}Q_{\mathrm{k},i} \qquad 6.10\mathrm{a}$$

$$\sum_{j\geq 1}\xi\gamma_{\mathrm{G},j}G_{\mathrm{k},j} + \gamma_{\mathrm{p}}P + \gamma_{\mathrm{Q},1}Q_{\mathrm{k},1} + \sum_{i>1}\gamma_{\mathrm{Q},i}\psi_{0,i}Q_{\mathrm{k},i} \qquad 6.10\mathrm{b}$$

The first of these two expressions effectively treats all variable actions as accompanying the permanent action (and thus applies ψ_0 to all variable actions) while the second considers the leading variable action as the primary action and allows a modest reduction in the design value of the permanent action.

Although EN 1990 permits the use of equations (6.10a) and (6.10b) as an alternative to (6.10), the National Annex may give guidance regarding the combination that should be used.

Recommended values of the partial factors and factors on accompanying actions are given in EN 1990 but these are confirmed or varied by the Nationally Determined Parameters (NDP) in the National Annex.

If the recommended values of NDP are adopted, it will be found that the option of using expressions (6.10a)/(6.10b) is generally less onerous than using (6.10).

It will also be found that, apart from storage areas, (6.10b) is the more onerous of (6.10a) and (6.10b), unless the permanent action is much (4,5 times) greater than the variable action. This is most unlikely in a multi-storey framed building.

Three types of combination of actions at the serviceability limit state are considered – characteristic, frequent and quasi-permanent. Expressions for these are given in 6.14b, 6.15b and 6.16b, as follows:

$$\sum_{j\geq 1} \mathrm{G}_{\mathrm{k},j} + P + Q_{\mathrm{k},1} + \sum_{i>1} \psi_{0,i} Q_{\mathrm{k},i} \qquad 6.14\mathrm{b}$$

$$\sum_{j\geq 1} G_{\mathrm{k},j} + P + \psi_{1,1} Q_{\mathrm{k},1} + \sum_{i>1} \psi_{2,i} Q_{\mathrm{k},i} \qquad 6.15\mathrm{b}$$

$$\sum_{j\geq 1} G_{\mathrm{k},j} + P + \sum_{i\geq 1} \psi_{2,i} Q_{\mathrm{k},i} \qquad 6.16\mathrm{b}$$

It is implicit in these expressions that partial factors are equal to unity. The factors for accompanying actions (ψ_0, ψ_1 and ψ_2) are given in EN 1990[5], but the National Annex may give additional information as to what values should be used. These values are specific for the type of load being considered, i.e. ψ_1 for snow is different from ψ_1 for wind.

For braced multi-storey building frames, the serviceability limit states to be considered will normally be those for the vertical and horizontal deflections of the frame and the dynamic performance of the floors. Crack widths may also need to be controlled for durability reasons in some situations (such as in car parks) and occasionally for appearance reasons. Guidance is given in EN 1992-1-1[7] and in *Multi-storey steel buildings. Part 3: Actions*[6].

3.3 Analysis for gravity loads

With the assumption of pinned behaviour for beam/column connections, all floor systems adopted for multi-storey buildings are statically determinate. Simple load allocation may be adopted to determine the governing moments, shears and axial forces in all elements: floors slabs, secondary beams, primary beams, columns and connections.

EN 1991-1-1, § 6.2.1(4) defines the reduction factor, α_A, that may be applied to gravity loads on floors, beams and roofs according to the area supported by the appropriate member.

§ 6.2.2(2) defines an equivalent factor, α_n, for gravity loads on walls and columns, depending on the number of storeys loading the appropriate element.

Not all imposed gravity loads qualify for the reduction. For example, it would not be appropriate where:

- Loads have been specifically determined from knowledge of the proposed use

- Loads are due to plant or machinery
- Loads are due to storage.

Appropriate account must be taken of the eccentricity of the line of shear through the simple beam to column connections. Figure 3.1 and Table 3.1 show the nominal moments that have traditionally been considered in different European countries. It is recommended that eccentricities are adopted that align with traditional practice in the country concerned, in order to ease the regulatory approval process. The moments are low and have only a modest implication in final column design.

As a further concession to simplicity, designers are not required to consider pattern live loading effects in simple construction.

Moments are not introduced into the column when the column is subject to symmetrical reactions and the column is therefore designed for axial force alone. Often, only columns on the edge of the structure will have unbalanced reactions. Most columns within a regular column grid will be designed for axial force only.

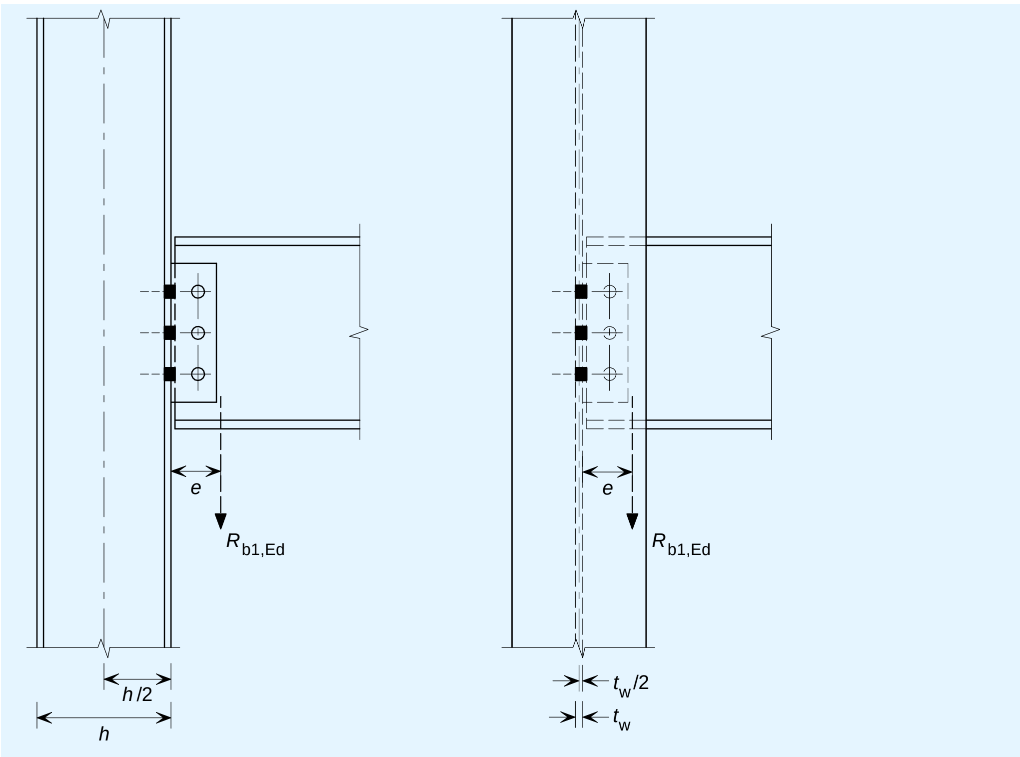
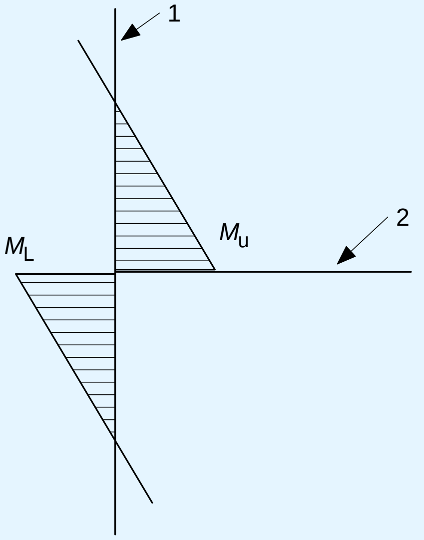


Figure 3.1 Nominal moments from floor beams

Table 3.1 Nominal Values of eccentricity '*e*' typically used for 'simple construction' in different European Countries

Country	Major axis eccentricity	Minor Axis eccentricity
Belgium	*h*/2	0
Netherlands	*h*/2	0
Germany	*h*/2	0
France	*h*/2	0
Spain	*h*/2	0
Italy	*h*/2	0

The nominal moments may be shared equally between the upper and lower columns, provided the ratio of their stiffness (I/L) does not exceed 1,5[4]. Outside this limit, the moments should be distributed in proportion to column stiffness.



$$\frac{M_u}{M_L} = \frac{I_{upper} / L_{upper}}{I_{lower} / L_{lower}}$$

But $M_{upper} = M_{lower}$ when $0{,}67 < \dfrac{I_{upper}/L_{upper}}{I_{lower}/L_{lower}} < 1{,}5$

1 Column
2 Floor beam

Figure 3.2 Distribution of nominal moments from floor beams

3.4 Allowance for second order effects

There are two options for low and medium rise frames and the allowance of second order effects.

3.4.1 Ensure α_{cr} > 10

For small scale construction of up to three storeys, it may be appropriate to ensure α_{cr} is greater than 10 by applying the simplified approaches of Section 2.6 of this document.

3.4.2 Design for 3 < α_{cr} < 10

More generally, it is appropriate to design the horizontal bracing structure for strength. It is then necessary to take appropriate account of second order effects in accordance with § 2.4.2(2) of EN 1993-1-1. The steps described in Section 2.5 should be followed.

3.5 Design Summary

References are to EN 1990-1-1[5].

- Use the least favourable of Equations 6.10a and 6.10b (where allowed by the National Annex) for the combination of actions for the ultimate limit state.
- Use Equations 6.14b, 6.15b, 6.16b for combinations of actions for the serviceability limit state, noting any recommendations in the National Annex.
- Use § 6.2.1(2) and (4) to determine the permissible reductions in variable actions applied to large areas.
- Carry out the analysis in accordance with Section 3.3 of this publication for gravity loads and 3.4 to assess the significance of second order effects and allow for them if necessary.

4 SERVICEABILITY LIMIT STATE

4.1 General

EN 1990[5], § 3.4 and 6.5 and EN 1993-1-1[1], § 7 require structures to satisfy the Serviceability Limit State. Criteria relevant to multi-storey buildings are:

- Horizontal deflections
- Vertical deflections on floor systems
- Dynamic response.

The general philosophy of the Eurocodes is not to offer prescribed general limits for horizontal and vertical deflections, but to recommend that limits should be specified for each project and agreed with the client. They acknowledge that National Annexes may specify relevant limits for general applications in specific countries.

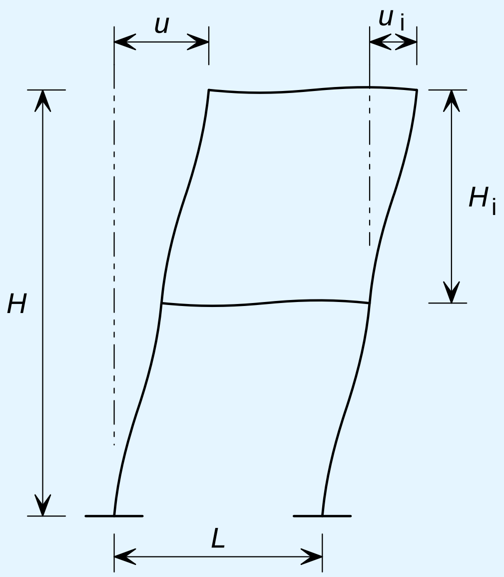
Sections 4.3 and 4.4 provide the definition of horizontal and vertical deflections and suggest some limits, based on Access Steel document SN034a[4].

4.2 Load combinations

As discussed in *Multi-storey steel buildings. Part 3: Actions*[6], different combinations of actions are used for serviceability and ultimate limit states. It is noteworthy that some countries only apply limits to response to variable actions (i.e. deflections due to permanent actions are not limited).

4.3 Horizontal deflection limits

The definitions of horizontal deflections in Annex A1 to EN 1990[5] are shown in Figure 4.10. Table 4.1 summarises typical horizontal deflection limits used in Europe.



Notation:
u Overall horizontal displacement over the building height H.
u_i Horizontal displacement over a storey height H_i

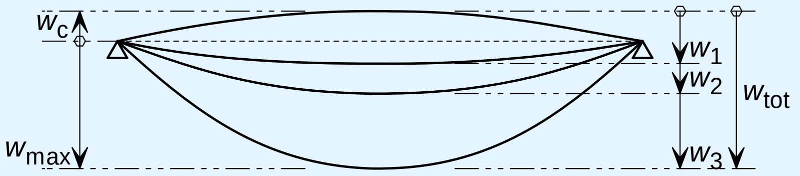
Figure 4.1 Definitions of horizontal deflections

Table 4.1 Horizontal deflection limits

Country	Deflection limits		Comments
	u	***u*****i**	
France			
Multi-storey buildings	$H/300$	$H_i/250$	These values should be verified against the deflections calculated from the characteristic combination, unless otherwise agreed with the client. The limit given for u applies for $H \leq 30$ m.
Germany			
There are no national deflection limits. The limits should be taken from manufacturers' instructions (technical approvals) or agreed with the client.			
Spain			
Multi-storey buildings:			These values are given in the national technical document for steel structures[8] and in the Technical Building Code[9] and should be used unless otherwise agreed with the client.
In general	$H/500$	$H_i/300$	
With fragile partition walls, facades, envelopes or rigid floor finishing elements		$H_i/500$	
High rise slender buildings (up to 100 m).	$H/600$		

4.4 Vertical deflection limits

The definitions of vertical deflections in Annex A1 to EN 1990[5] are shown in Figure 4.20.



w_c is the precamber in the unloaded structural member
w_1 is the initial part of the deflection under the permanent loads on the relevant combinations of actions
w_2 is the long-term part of the deflection under permanent loads
w_3 is the additional part of the deflection due to the variable actions of the relevant combinations of actions
w_{tot} is the total deflection as the sum of w_1, w_2, w_3
w_{max} is the remaining total deflection taking into account the precamber

Figure 4.2 Definitions of vertical deflections

Table 4.2 summarises typical vertical deflection limits used in Europe.

Table 4.2 Vertical deflection limits

Country	Deflection limits w_{max}	Deflection limits w_2+w_3	Comments
France			
Roof, in general	*L*/200	*L*/250	These values are given in the French National Annex to EN 1993-1-1 and should be used unless otherwise agreed with the client. The values of the deflections calculated from the characteristic combinations should be compared to these limits.
Roofs frequently carrying personnel other than for maintenance	*L*/200	*L*/300	
Floors, in general	*L*/200	*L*/300	
Floors and roofs supporting plaster or other brittle toppings or non-flexible parts	*L*/250	*L*/350	
Floors supporting columns (unless the deflection has been included in the global analysis for the ultimate limit state)	*L*/400	*L*/500	
When w_{max} can affect the appearance of the building	*L*/250	-	
Germany			
There are no national deflection limits. The limits should be taken from manufacturers' instructions (technical approvals) or should be agreed with the client.			
Spain			
Roofs, with access for maintenance only	-	*L*/250	
Roofs in general		*L*/300	
Beams and floors, without fragile elements		*L*/300	
Beams and floors, supporting ordinary partition walls and rigid floor finishing elements with expansion joints		*L*/400	
Beams and floors, supporting fragile elements such as partition walls, facades envelopes or rigid floor finishing elements		*L*/500	
Beams supporting columns		*L*/500	
Beams supporting masonry walls		*L*/1000	

4.5 Precambering

Deflections under permanent loads may be a significant part of the overall deflection of floor beams. This is particularly true for composite floor systems that are constructed without propping (as is recommended for fast, simple construction).

In such cases, designers should specify pre-cambering to ensure that the primary and secondary floor systems are flat and level once the structure has been completed. Annex A1 of EN 1990[5] makes specific provision for recognising the benefits of precambering.

4.6 Dynamic response

Annex A.1.4.4 of EN 1990 states the following requirements for the dynamic response of all structures.

> **Vibrations**
> *(1) To achieve satisfactory vibration behaviour of buildings and their structural members under serviceability conditions, the following aspects, amongst others, should be considered:*
>
> *a) the comfort of the user;*
>
> *b) the functioning of the structure or its structural members (e.g. cracks in partitions, damage to cladding, sensitivity of building contents to vibrations).*
>
> *Other aspects should be considered for each project and agreed with the client.*
>
> *(2) For the serviceability limit state of a structure or a structural member not to be exceeded when subjected to vibrations, the natural frequency of vibrations of the structure or structural member should be kept above appropriate values which depend upon the function of the building and the source of the vibration, and agreed with the client and/or the relevant authority.*
>
> *(3) If the natural frequency of vibrations of the structure is lower than the appropriate value, a more refined analysis of the dynamic response of the structure, including the consideration of damping, should be performed.*
>
> *Note: for further guidance, see EN 1990-1-1, EN 1990-1-4 and ISO 10137*
>
> *(4) Possible sources of vibration that should be considered include walking, synchronised movements of people, machinery, ground borne vibrations from traffic, and wind actions. These, and other source, should be specified for each project and agreed with the client.*

In practice, for low and medium rise buildings for commercial or residential use, the key issue is the dynamic response of the floor system to human excitation, primarily either from walking or from a single heavy ‘foot-fall’.

The dynamic response of floor systems to human excitation is complex, for three reasons:

- The nature of the excitation is uncertain in magnitude, duration and frequency of occurrence.
- The structural response is substantially influenced by the magnitude of the damping in the structure and the damping effect of non-structural components of the building and its fittings, furnishings and furniture.
- Human perception of vibration and the definition of appropriate acceptance criteria are both very imprecise, varying between individuals and for a single individual over time.

It must be emphasised that a floor that has a 'lively' response to human excitation is most unlikely to have inadequate or impaired strength. Traditional timber floors have always exhibited such behaviour yet have performed satisfactorily. However, both the increasing use of longer span steel floor systems and the move to lighter construction increase the probability that performance may cause discomfort to some users. Designers therefore need to pay attention to this aspect of serviceability.

Historically, designers have used the natural frequency of the floor as the sole measure of acceptable performance. A sufficiently high natural frequency means that a floor is effectively 'tuned' out of the frequency range of the first harmonic component of walking. However, resonance might still occur with higher harmonics. As a guideline, a fundamental frequency above 4 Hz is usually appropriate, but no requirements are given in EN 1994[2] and the designer should seek guidance in the national regulations.

Much more effective, though more complex, methods of assessing dynamic serviceability have emerged recently:

- *Vibration design of floors RF32-CT-2007-00033*: This is freely available from RWTH Aachen. It provides a single mode analysis on the floor.
- *Design of floors for vibration*: *A new Approach*[10] presents a more comprehensive, multi-mode approach to the same methods of assessment.

Design software is becoming available that takes full account of the method presented in Reference 10.

4.7 Design summary

- Verify the horizontal deflections defined in Section 4.3 against the criteria defined in the relevant National Annex
- Verify the relevant vertical deflection, defined in Section 4.4 against the criteria defined in the relevant National Annex
- Consider pre-cambering for beams greater than 10 m in length
- Verify the dynamic response of the floor against one of the references given in Section 4.6.

5 ULTIMATE LIMIT STATE

5.1 Introduction

Design for the Ultimate Limit State, i.e. verification of the strength of all the structural components of the building to resist the actions identified by the global analysis, remains the core of the detailed design process.

Fortunately, many design aids are now available to assist designers; these have relieved them of much of the detailed effort that was previously required. The following sections provide comprehensive guidance on how to take full account of these aids, while still providing reference back to their basis in the Eurocodes.

5.2 Floor systems

5.2.1 Floor slab

Either a composite slab or a precast floor will have been chosen during conceptual design. Both may be designed from first principles but this is rarely, if ever, done in practice.

A composite slab may be designed to EN 1993-1-3[1] for the construction condition, EN 1994-1-1[2] for the completed structure and EN 1994-1-2 for the fire condition.

A precast reinforced concrete floor may be designed to EN 1992-1-1[7].

All these standards make provision for design assisted by testing, in accordance with Annex D of EN 1990[5].

For such specialist construction products with wide application in practice, the design assisted by testing route has been adopted by the manufacturers because it offers greater design resistance than that determined by calculation. Manufacturers supplying into a national market will usually offer appropriate design tables which take full account of Nationally Determined Parameters defined in relevant National Annexes.

Designers should use these design tables wherever they are available.

5.2.2 Downstand non-composite beams

Downstand non-composite floor beams are used to support precast floor systems, and, possibly, composite floor slabs that are not attached to their supporting beams by shear connectors. Downstand roof beams generally carry the purlins that support roof systems

Depending on the details of construction, these beams may be:

- Fully restrained for both construction and in-service conditions.
- Restrained at points of load application for both construction and in-service conditions.

- Unrestrained for the construction condition and either:
    - Fully restrained for the in-service condition (floor beams supporting precast planks)
    - Restrained at points of load application for the in-service condition
    - Unrestrained for both construction and in-service conditions.

Depending on the slenderness of the beams, there may be a substantial difference of resistance between restrained and unrestrained conditions. It follows that, where the restraint condition improves when the structure is fully constructed, separate verifications should be carried out for both construction and in-service stages.

The detailed design process is presented in Figure 5.1 to Figure 5.4. It is possible to cover all the cases listed above by appropriate consideration of the restraint conditions and hence the design buckling resistance described in Figure 5.4.

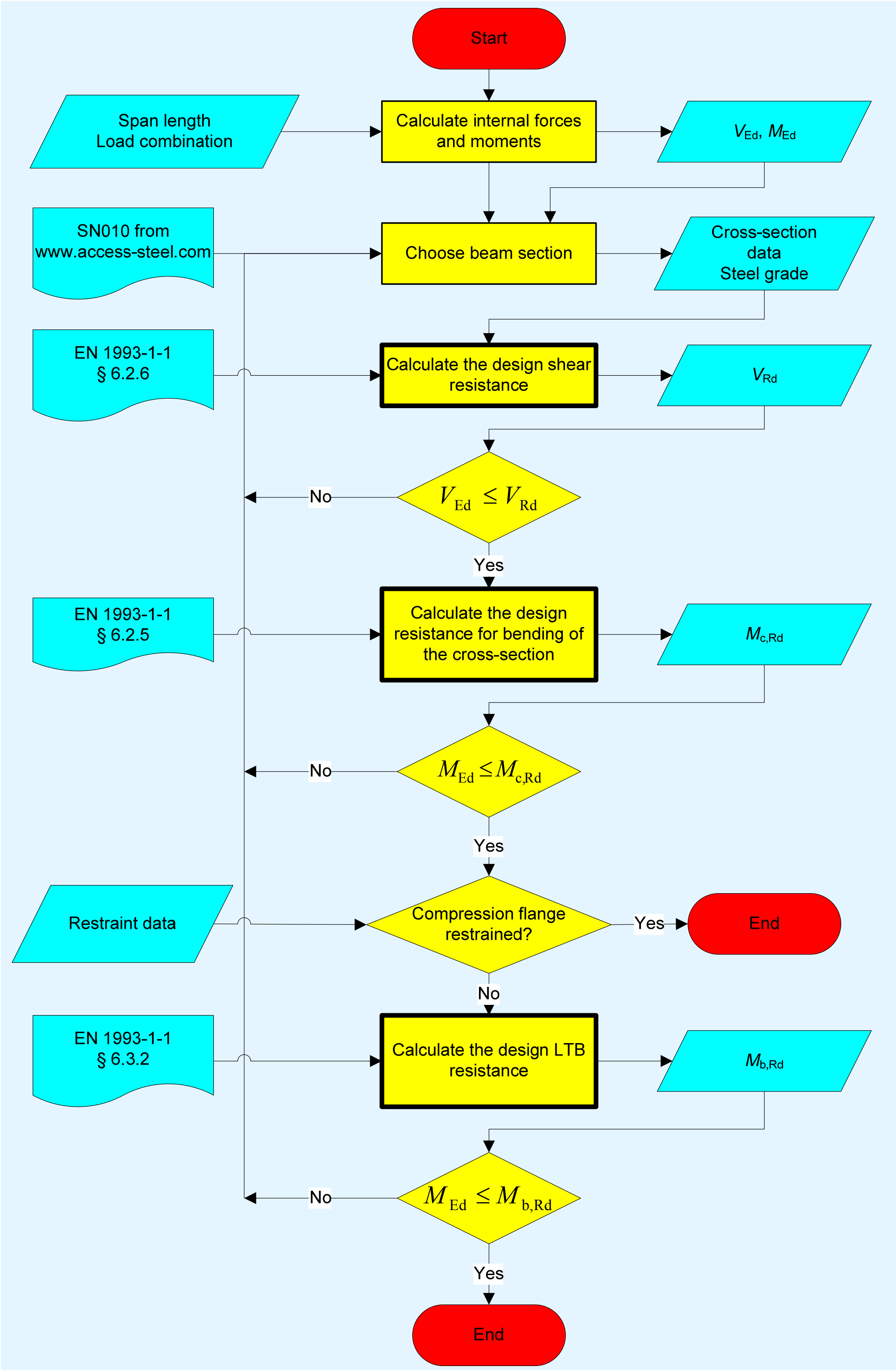


Figure 5.1 Overall procedure for the design of a non-composite beam under uniform loading

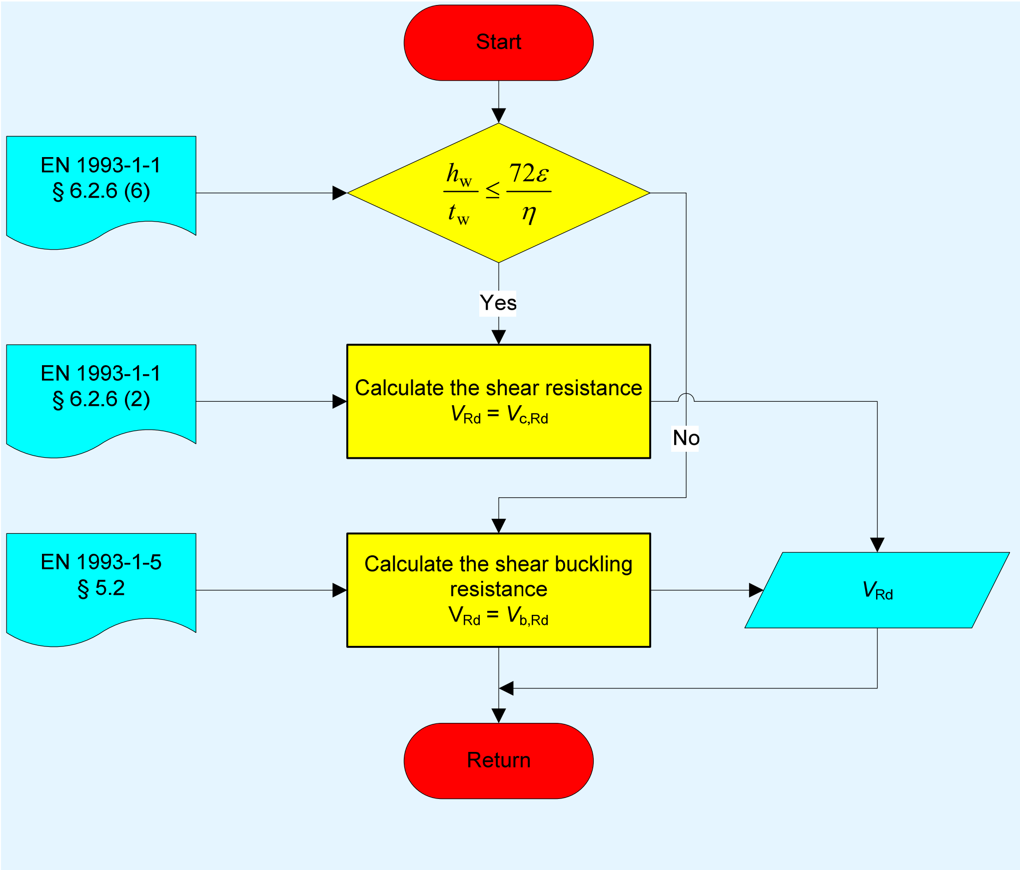


Figure 5.2 Detailed procedure for determining the design shear resistance of a beam

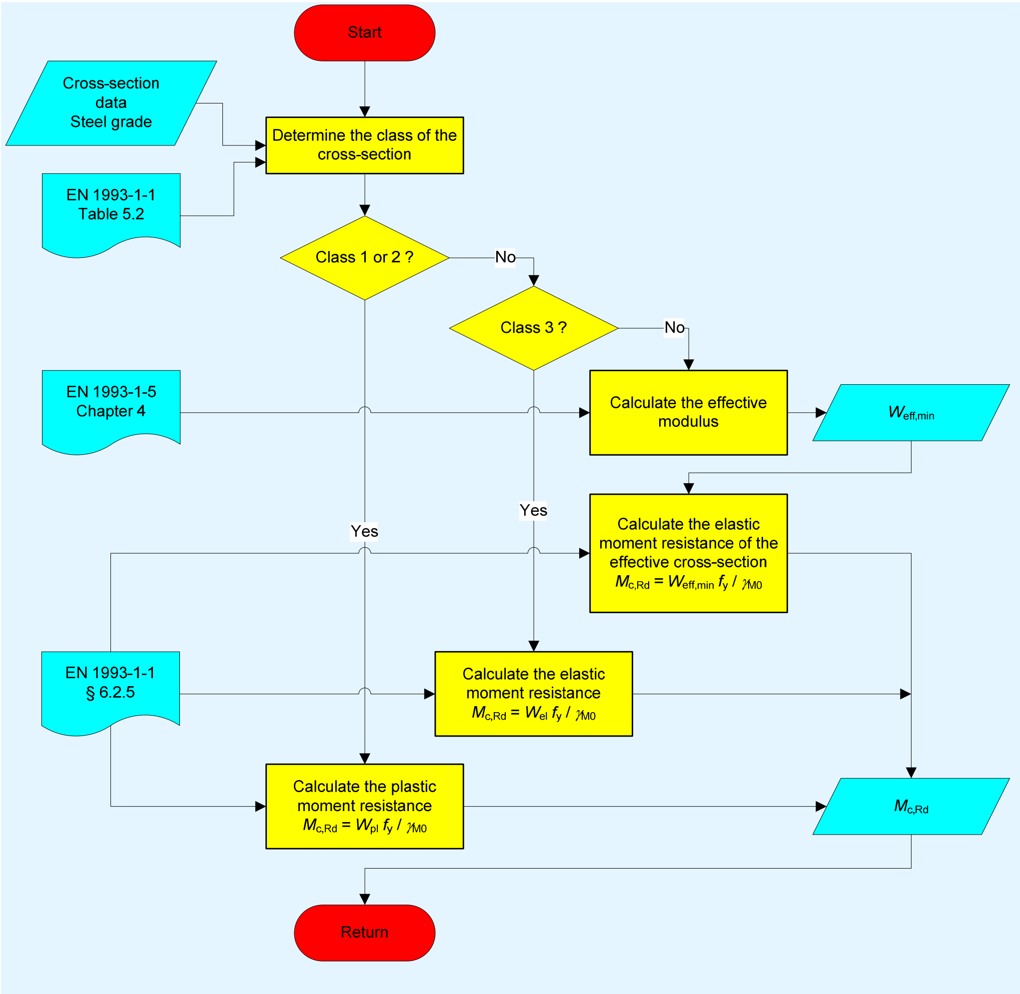


Figure 5.3 Detailed procedure for determining the design resistance to bending of a beam cross-section

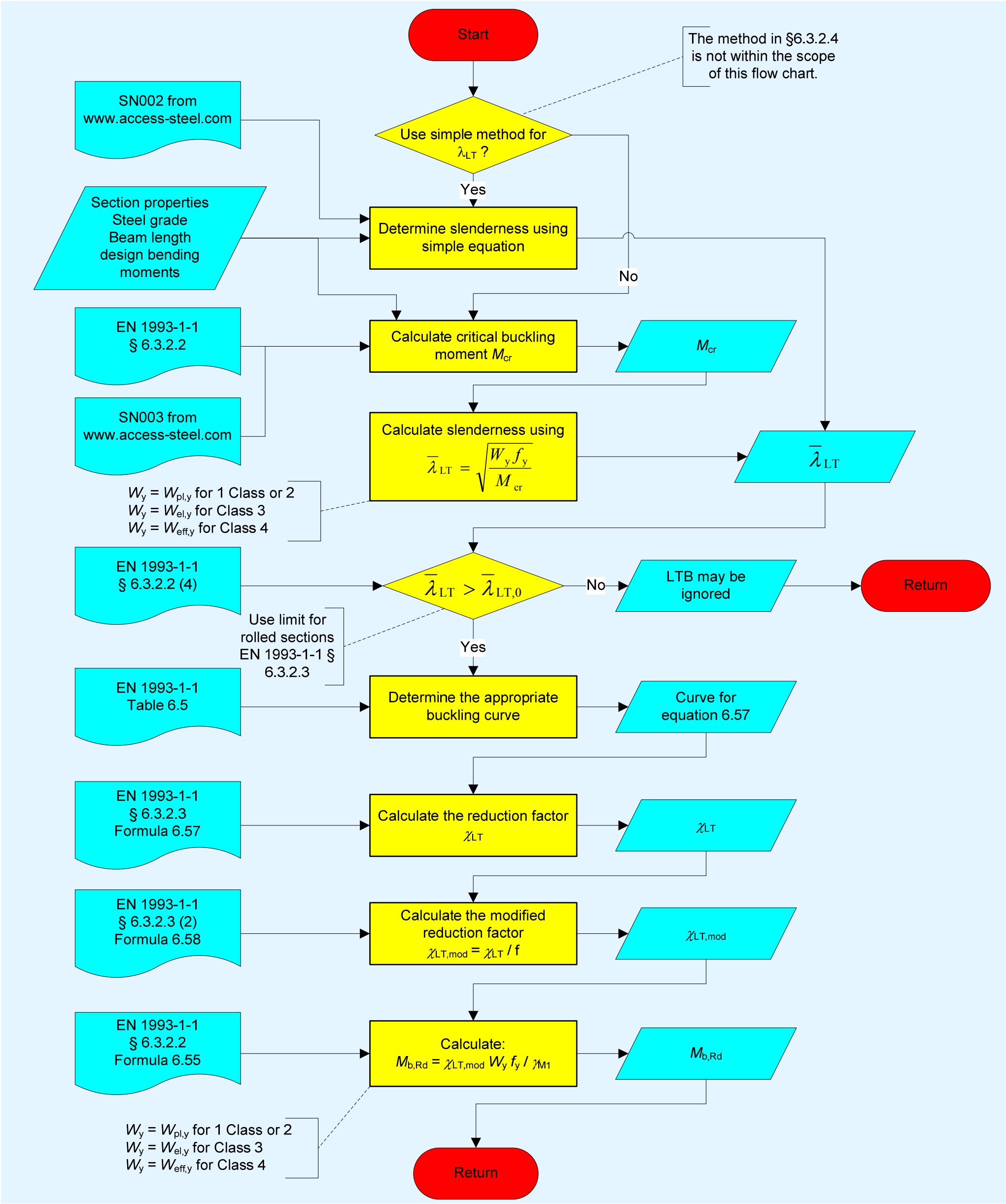


Figure 5.4 Detailed procedure for determining the design lateral-torsional buckling resistance of a beam

Numerous worked examples are available to demonstrate these design processes from first principles.

Appendix A to this publication presents:

- WE1 A simply supported, laterally restrained beam.
- WE2 A simply supported beam with intermediate lateral restraints.

Other examples can be found at the Access Steel web site[4]. These include two interactive worked examples, where users may input their own variables to carry out a worked example to their specification.

Use of design aids and software

The member resistance calculator provided in the spreadsheet accompanying *Multi-storey steel buildings. Part 8 Design software – section capacity*[11] may be used to calculate the resistance of members in compression, bending, tension and combined bending and compression, as well as the resistance of webs and the shear resistance of cross sections.

Design software is increasingly available for the design of both restrained and unrestrained beams and should generally be adopted for commercial design.

5.2.3 Downstand composite beams

Downstand composite beams are used as:

- Secondary beams, to provide direct support to composite floor slabs, with shear connectors to ensure overall composite action.
- Primary beams, to support the secondary beams and transmit their loads to the columns. Shear connectors are used to ensure overall composite action.

For simplicity in construction, it is strongly recommended that composite beams are designed to be unpropped for the construction condition. Verification for this condition needs to be carried out in accordance with Section 5.2.2 of this guide before proceeding with the composite condition for the completed structure.

Before proceeding with the composite checks, it is necessary to determine the approach to shear connection (this may well have been considered initially during the conceptual design stage). Two approaches are possible:

Full interaction

Sufficient shear connection is provided to develop the full plastic resistance of the composite section. This approach simplifies the design approach and maximises the stiffness of the composite beam. However, if the beam is larger than is required for the ultimate limit state of the completed structure, significantly more shear connectors may be required than would be necessary for basic strength. There is clearly a cost implication for these additional shear connectors, particularly for longer span primary beams. It may also be difficult, or impossible, to fit sufficient shear connectors onto the top flange. Figure 5.5 presents the detailed design process for full shear connection for secondary beams (primary beams are likely to be designed for partial interaction). This simplified approach is restricted to Class 1 or 2 sections; this is unlikely to be restrictive in practice.

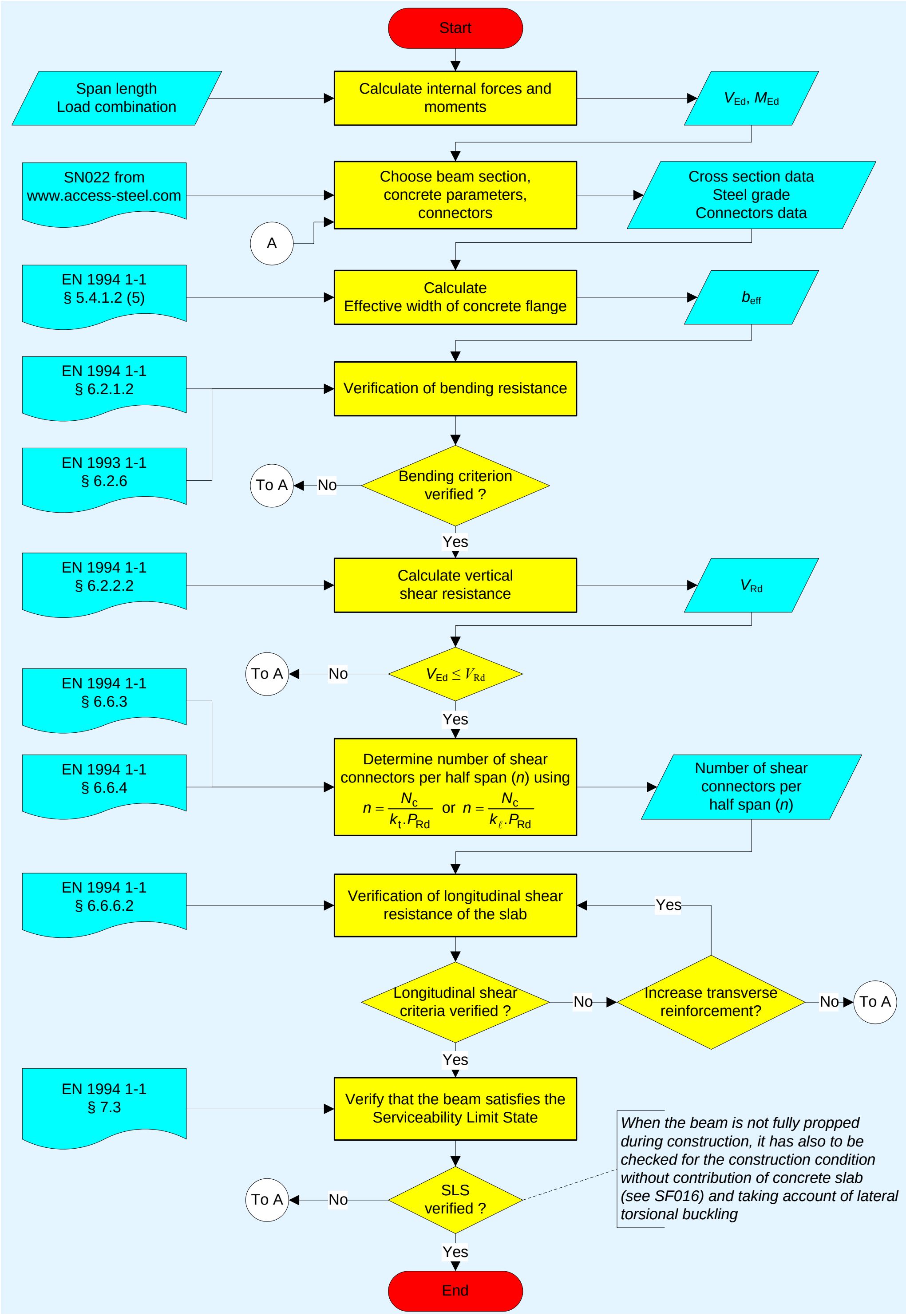
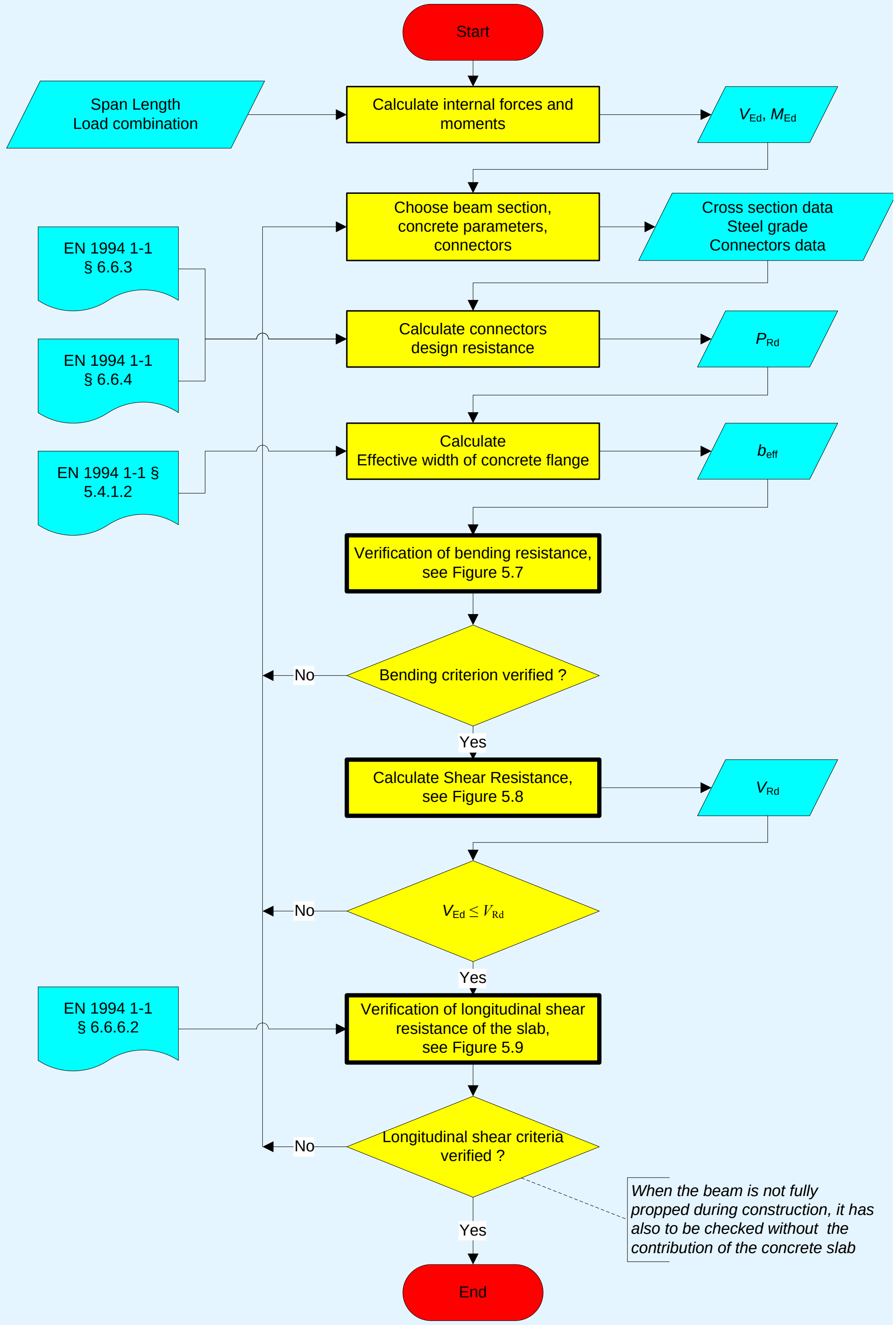


Figure 5.5 Design of simply supported composite beams with full shear connection and Class 1 or 2 steel beams

Partial interaction

Where the size of the steel beam is defined either by the unpropped construction condition or by the serviceability of the completed structure, it will have excess resistance for the ultimate limit state of the composite section. In such cases, adopting partial shear connection is likely to be more economic.

In this case, Figure 5.6 to Figure 5.9 present the overall procedure and detailed sub-processes for design.



Note that rectangular boxes with heavy surrounds are references to detailed sub-processes in Figure 5.7 to Figure 5.9.

Figure 5.6 Overall procedure for the design of a simply supported composite beam

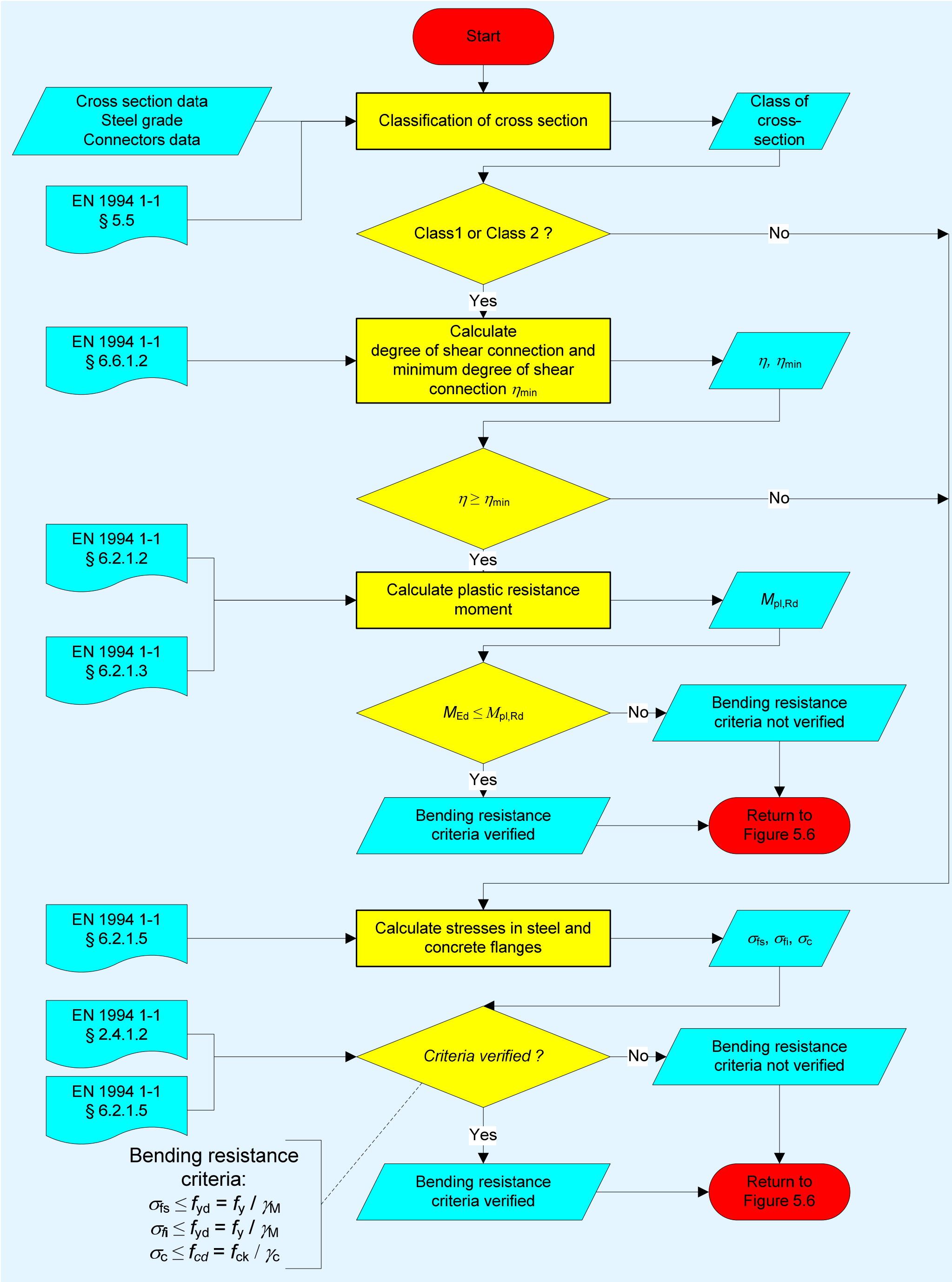


Figure 5.7 Verification of bending resistance of composite beam

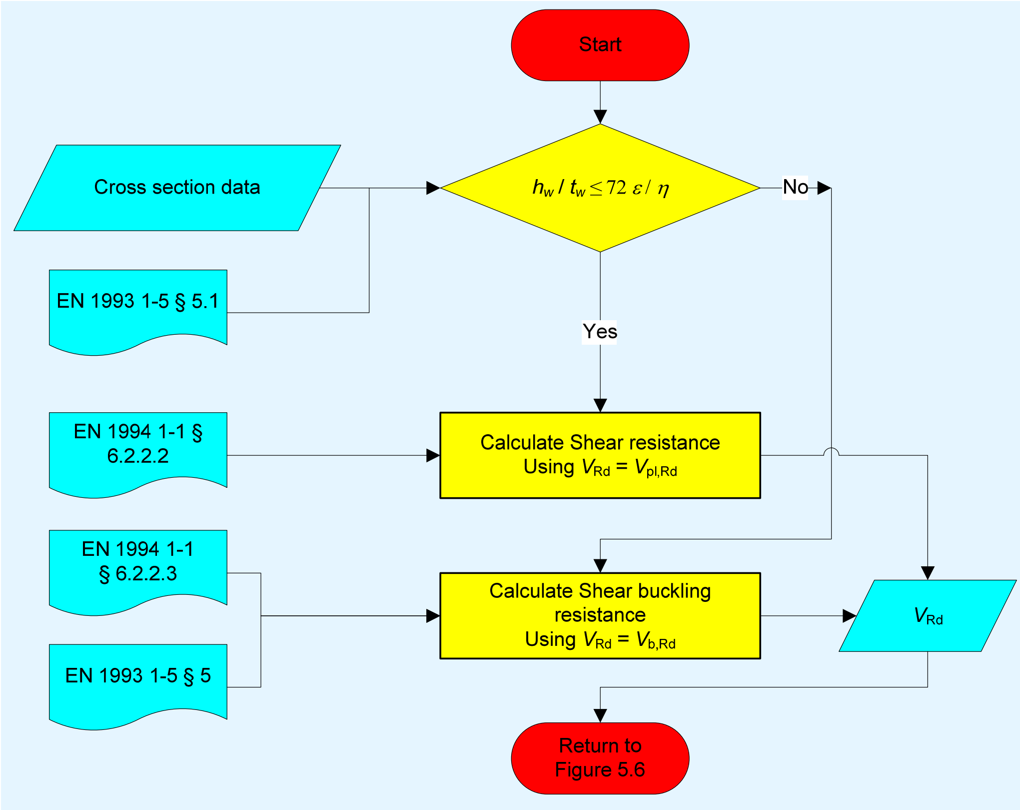


Figure 5.8 Verification of vertical shear resistance

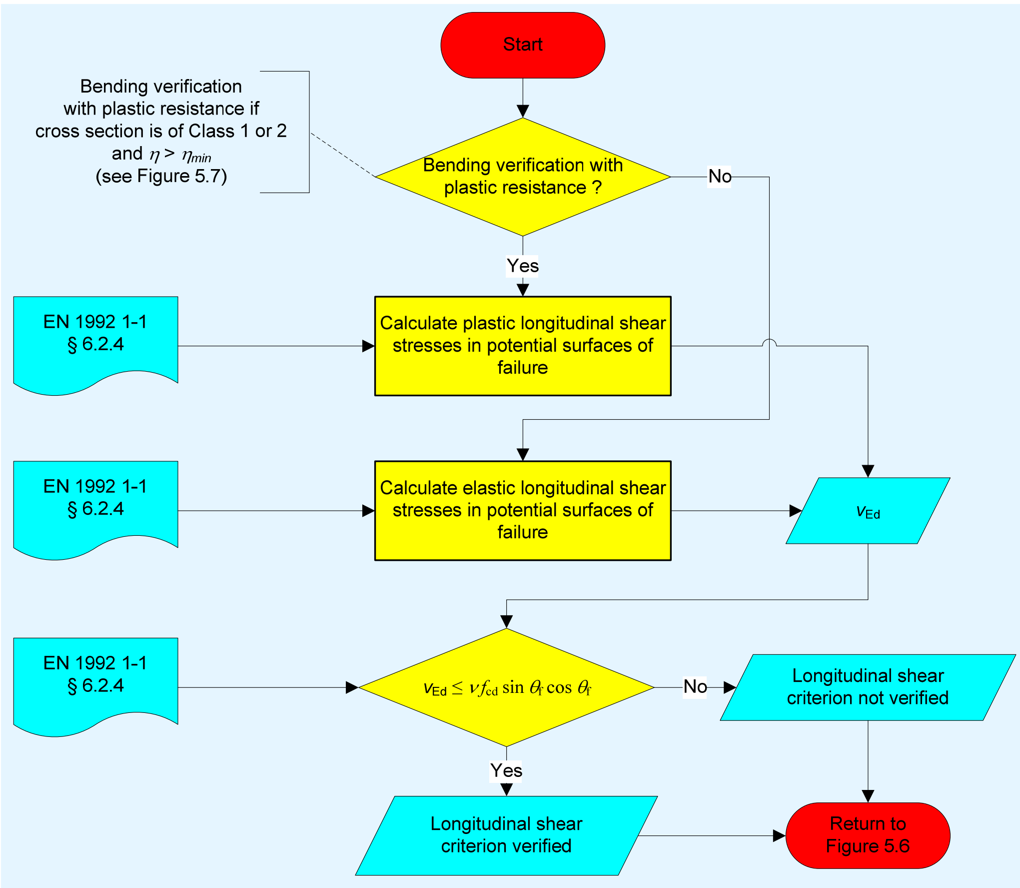


Figure 5.9 Verification of longitudinal shear resistance of the slab

Worked examples are provided in Appendix A to demonstrate the detailed design of a secondary and primary composite beam.

Shear connection resistance

In the foregoing verifications of the resistance of composite beams, it should be noted that shear connection resistance is a function of:

- The proportions of the deck through which the stud is welded.
- The position of the stud(s) with the troughs of the deck
- The number of studs within a single trough.

Deck manufacturers' literature should be consulted to determine appropriate design values.

Appendix A to this publication presents:

- WE3 Simply supported secondary composite beam (with partial shear connection)
- WE4 Simply supported primary composite beam

Use of design aids and design software

With the number of variables to be considered in composite beams, it is not practical to provide any form of tabular design aid. However, a software specification for a composite beam program has been prepared as a separate part of this publication.

Composite beam software is already available in some major European Markets.

In addition, there are two interactive worked examples on the Access Steel web site[4] that address simply supported secondary and primary beams. Users may input their own variables to carry out a worked example to their specification.

5.2.4 Integrated floor beams

Integrated floor beams, which are principally encased within the depth of the floor slabs, are manufactured by several suppliers. They are all supported by manufacturers' design data and some are supported by specialist software.

Manufacturers' design data or software should be used in practice.

5.2.5 Cellular beams

Cellular beams are a specialist form of downstand beam where large openings in the beam webs enable services to share the same depth as the structure within the floor zone.

There are several manufacturers of such products, who have all developed specialist design approaches for 'cold' design and some have extended their approaches to encompass the fire limit state. All such software complies with the Eurocodes.

Such beams are therefore always designed to a performance specification for a specific project, using specialist software. Design from first principles to the Eurocodes is not practicable.

5.3 Columns

As discussed in Section 3, one of the advantages of simple construction is that internal columns in regular grids may generally be designed for axial load only.

External columns will be subject to both axial loads and moments from connection eccentricity.

A simple design method for columns in simple construction with nominal moments from connection eccentricity is described in Section 5.3.2.

5.3.1 Columns subject to axial load only

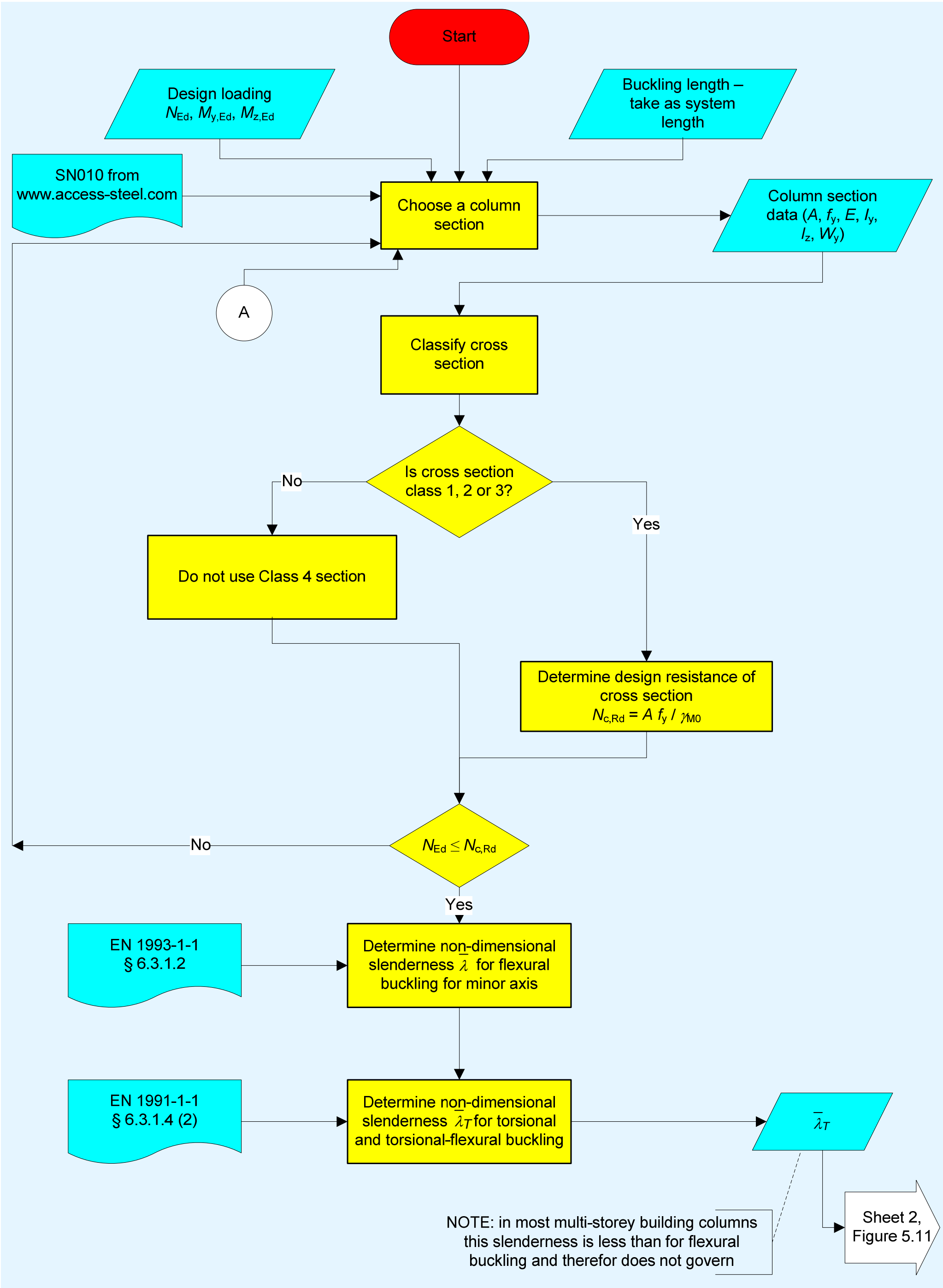


Figure 5.10 Verification of column resistance – sheet 1

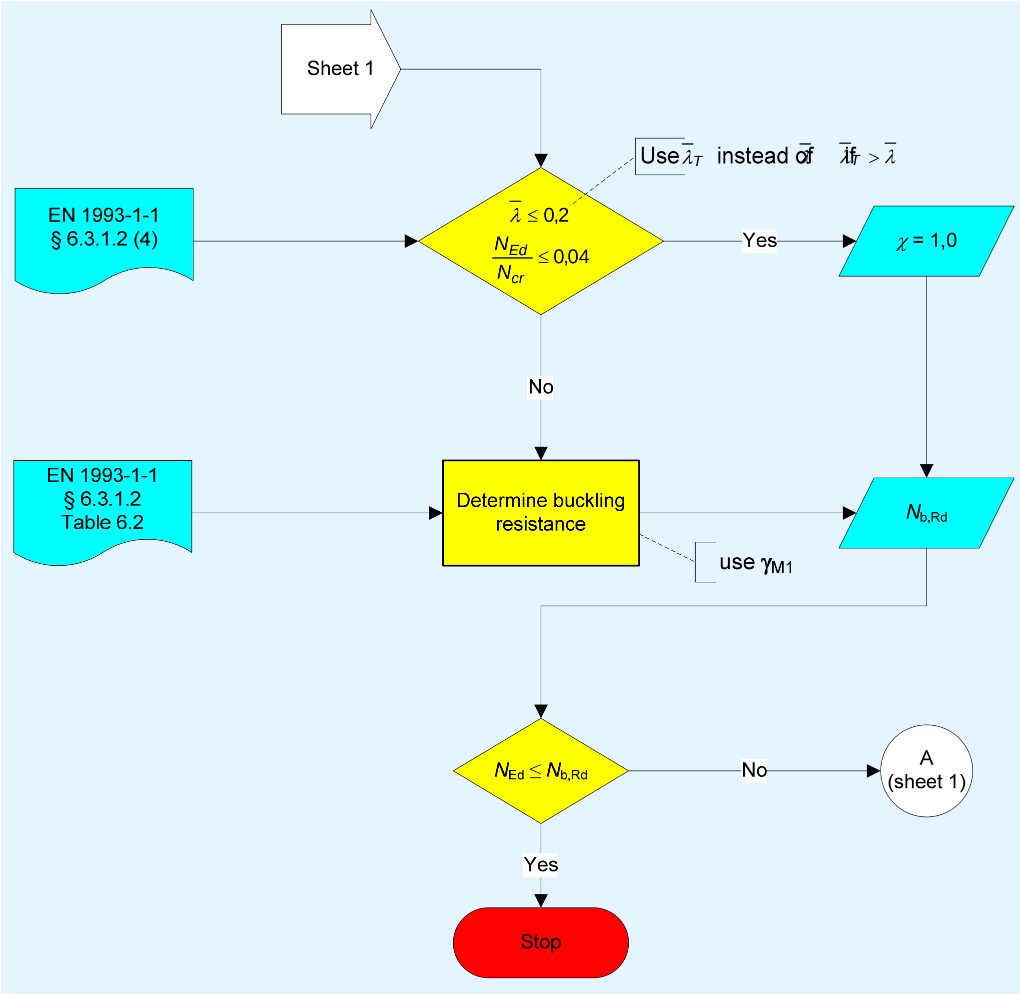


Figure 5.11 Verification of column resistance – sheet 2

Figure 5.10 and Figure 5.11 describe the detailed design verifications that are necessary to verify a column subject to axial load only.

The application of this process is illustrated in WE5 in Appendix A.

Use of design aids

The member resistance calculator provided in *Multi-storey steel buildings. Part 8: Design software – section capacity*[11] may be used to calculate the resistance of members in compression, bending, tension and combined bending and compression.

In addition, Access Steel document SI004[4] provides an interactive worked example where users may input their own parameters to achieve the same result.

5.3.2 Columns subject to axial load and moments

The general methods in EN 1993-1-1[1] for the design of members subject to axial force and moments are complex for H columns in low and medium rise buildings. Access Steel document SN048 (only available in English) of provides an NCCI for such cases, including a justification of its simplifications. The process to be adopted is described below:

Scope

It may only be adopted where:

- The column is a hot rolled I or H section.
- The cross-section is Class 1, 2 or 3 under compression.
- The bending moment diagrams about each axis are linear.
- The column is restrained laterally in both the y and z directions at each floor level but is unrestrained between the floors.
- The buckling length is the same in both directions.

Design criterion: Overall buckling

The column must satisfy the single interactive equation:

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{b,min,Rd}}} + \frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{b,Rd}}} + 1{,}5\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{cb,z,Rd}}} \leq 1{,}0 \qquad 5.1$$

where:

$N_{\mathrm{b,min,Rd}}$ is the lesser of $N_{\mathrm{b,y,Rd}}$ and $N_{\mathrm{b,z,Rd}}$

$N_{\mathrm{b,y,Rd}}$ and $N_{\mathrm{b,z,Rd}}$ are the flexural buckling resistances about the y and the z axes

$M_{\mathrm{b,Rd}}$ is the lateral-torsional buckling resistance

$M_{\mathrm{cb,z,Rd}} = \dfrac{f_{\mathrm{y}}W_{\mathrm{pl.z}}}{\gamma_{\min}}$ for Class 1 and 2 sections

and $= \dfrac{f_{\mathrm{y}}W_{\mathrm{el,z}}}{\gamma_{\min}}$ for Class 3 sections

$\gamma_{\min} = \gamma_{\mathrm{M1}}$

It should be noted that this equation leads to a conservative answer when Annex B of EN 1993-1-1 is used but no study has been done regarding its use with Annex A of the same standard.

If this equation is not satisfied, then the more accurate expressions given in equations 6.61 and 6.62 of EN 1993-1-1 can still be used.

Design criterion: Local verification

For the lowest storey column, if the base is nominally pinned (as is usually the case), the axial force ratio must also satisfy:

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{b,y,Rd}}} < 0{,}83 \qquad 5.2$$

where:

$N_{\mathrm{b,y,Rd}}$ is resistance to buckling about the **major** axis

Figure 5.12 presents a flowchart to describe this simple procedure.

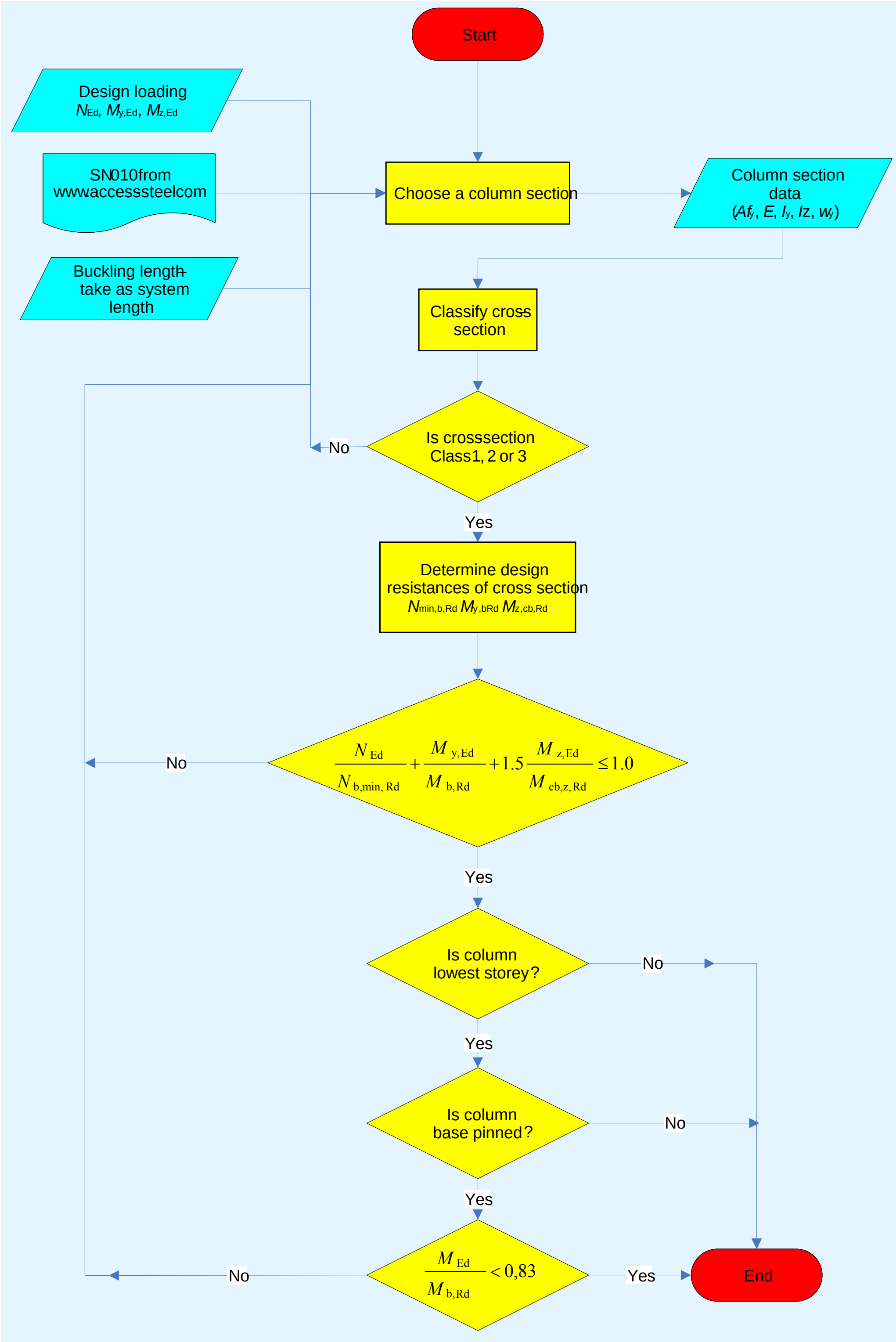


Figure 5.12 Simplified procedure for verification of column subject to axial load and nominal moments

Use of design aids

Design software is becoming increasingly available.

Because of the wide range of parameters involved, it is not feasible to use tabulated data for the final design. However, tabulated design data can be used to determine the denominators in Equations 5.1 and 5.2.

5.4 Vertical bracing

In a braced steel multi-storey building, the planes of vertical bracing are usually provided by diagonal bracing between two lines of columns, as shown in Figure 5.13.

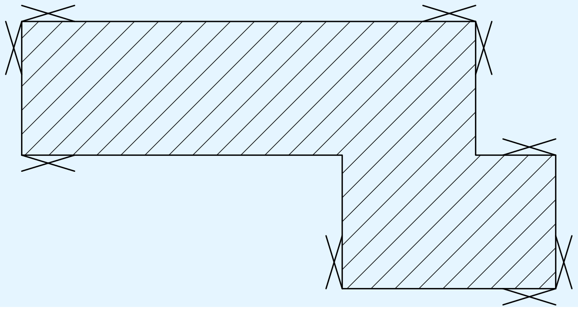


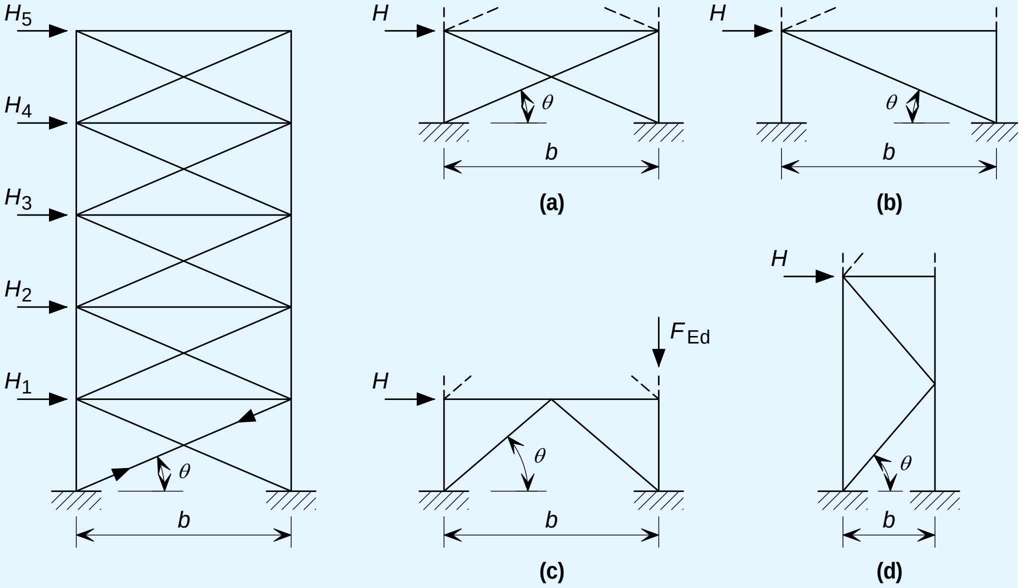
Figure 5.13 Typical positioning of vertical bracing

The vertical bracing must be designed to resist the forces due to the following:

- Wind loads.
- Equivalent horizontal forces, representing the effects of initial imperfections, Section 2.4.

These loads are amplified if necessary ($\alpha_{cr} < 10$) to allow for second order effects as described in Section 2.4.

Forces in the individual members of the bracing system must be determined for the appropriate combinations of actions (see Section 3.2). For bracing members, design forces at ULS due to the combination where wind load is the leading action are likely to be the most onerous.



At each floor level, $H_i = 0{,}025 \times V_{Ed,i}$ where $V_{Ed,i}$ is the total design load applied at that floor level

(a) Cross bracing
(b) Diagonal bracing
(c) Horizontal k bracing
(d) Vertical k bracing

Figure 5.14 Typical arrangements of vertical bracing (as Figure 2.3)

The design of the members in any bracing system is generally straightforward. However, the following specific points need to be addressed:

5.4.1 Tension only systems

Figure 5.14(a) shows nominally statically indeterminate systems with cross bracing. In practice, the diagonal members are likely to have a high slenderness (either as flats or as small angles to minimise intrusion into the building). The contribution from the compressive diagonal is therefore ignored.

5.4.2 Load reversal in statically determinate bracing systems

Figure 5.14 (b), (c) and (d) show examples of statically determinate bracing systems. The loading on most bracing systems is fully reversing. It is therefore only necessary to design the diagonals for the more critical condition, when the member is in compression.

5.4.3 Typical bracing members

Bracing members are connected using nominally pinned joints and therefore they carry axial loads only.

Bracing members can be welded or bolted to the main structural members. For bolted connections, use of normal (non-preloaded) bolts is generally appropriate for bracing in the scope of this guide (up to 12 storeys).

Typical sections used as bracing include flats, angles and channels.

Flat bar

Two calculations must be carried out to determine the tension resistance of flat bars:

- Gross section resistance, by using equation 6.6 of EN 1993-1-1. The partial factor to be used in this equation is γ_{M0}.
- Net section resistance, by using equation 6.7 of EN 1993-1-1. The partial factor to be used in this equation is γ_{M2}.

The compression resistance of flats should be carried out by following the method given in EN 1993-1-1, § 6.3.1.

Angles

When the bolts are positioned on the centroid of the section, the tension resistance of angles may be carried out as described for flat bars. When the bolts are positioned away from the centroid of the section the following procedures may be adopted.

Single line of bolts along the member

Where there is a single line of bolts along the angle and the bolts are not aligned with the centroid of the section, there is an additional bending moment due to the eccentricity. EN-1993-1-8, § 3.10.3 gives rules for the calculation of the tensile resistance in this case.

Multiple bolts across the member

Where there is more than one bolt across the section, EN 1993 does not give guidance to account for the eccentricity. In order to account for the additional bending moment the designer has two options:

- To use the interaction equations 6.61 and 6.62 of EN 1993-1-1
- To use other recognised sources of information, such as *Steel building design: Design data*[12], which provides an alternative method to account for this effect.

The compression resistance of angles should be calculated by using the method given in § 6.3.1 of EN 1993-1-1.

Where bolts are located away from the centroid of the section, the eccentricity will generate an additional bending moment on the member. As previously described for the tensile resistance of angles, this bending moment can be accounted for in two ways:

- By using the interaction equations 6.61 and 6.62 of EN 1993-1-1
- By calculating a modified slenderness as given in EN 1993-1-1, Annex BB 1.2 and applying it to the method given in § 6.3.1.

If the angle is welded instead of bolted, the forces distribute across the member and no bending effects needs to be considered.

When the bracing consists of unequal angles, it is important to specify which leg is connected.

Channels

The tension and compression resistances of channels are carried out in a similar way to that described for angles.

Channels are invariably connected through the web, either by welding or by means of bolts. This introduces an eccentricity with respect to the centroid of the section. Although EN 1993-1-1 does not explicitly allow the use of Annex BB 1.2 for the compression resistance of channels, the authors suggest that the approach may also be used for this purpose.

When channels with thin webs are used, bearing of the bolts on the channel may be critical. In order to avoid this problem the designer may specify larger bolts or the web of the channel may be thickened by welding a plate on the web.

5.5 Horizontal bracing

Horizontal bracing, or at least floor diaphragm action, is necessary to transmit horizontal forces and requirements for horizontal restraint to planes of vertical bracing.

Where triangulated steel bracing is adopted, the design approaches are essentially the same as those for vertical bracing. However, in general, it is more economical to use the floor as a diaphragm.

All floor solutions involving permanent formwork such as metal decking fixed by studs welded to the beams, with in-situ concrete infill, provide an excellent stiff diaphragm to carry horizontal forces to the bracing system.

Floor systems involving precast concrete planks require proper consideration to ensure adequate transfer of forces if they are to act as a diaphragm. The coefficient of friction between planks and steelwork may be as low as 0,1 and even lower if the steel is painted. This will allow the slabs to move relative to each other and to slide over the steelwork. Grouting between the slabs will only partially overcome this problem, and for large shears, a more positive tying system will be required between the slabs and from the slabs to the steelwork.

Connection between planks may be achieved by reinforcement in the topping. This may be welded mesh or ties may be placed along both ends of a set of planks to ensure the whole panel acts as one. Typically, a 10 mm bar at half depth of the topping will be satisfactory.

Connection to the steelwork may be achieved by one of two methods:

- Enclose the slabs by a steel frame (on shelf angles, or specially provided constraint) and fill the gap with concrete.
- Provide ties between the plank topping and a local topping to the steelwork (known as an 'edge strip'). The steel beam has some form of shear connectors to transfer forces between the in-situ edge strip and the steelwork.

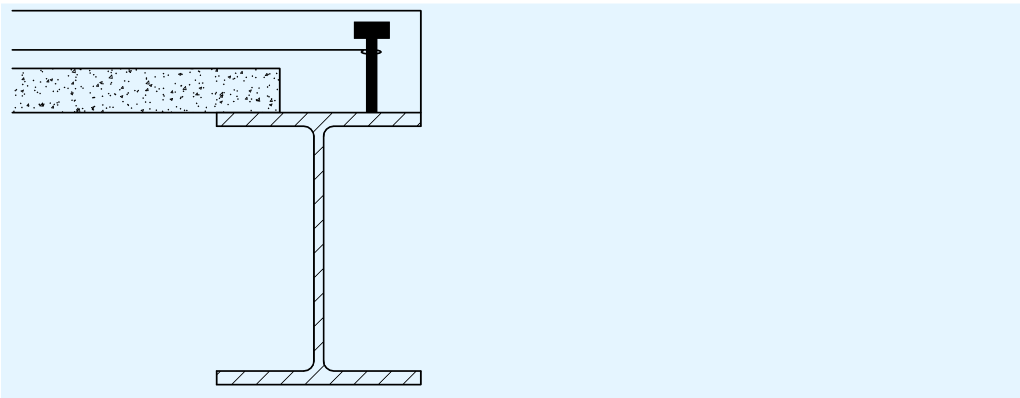


Figure 5.15 Possible connection between precast concrete plank and steelwork to ensure diaphragm action of the floor

Appropriate attention needs to be given to the 'load paths' that transfer the end shears of the horizontal diaphragms in the vertical bracing or concrete core. It is frequently possible to ensure that the 'end posts' of the horizontal diaphragms (or trusses) are also the top 'end posts' of the vertical bracing. Where concrete cores resist horizontal actions, it is usually possible to tie the concrete diaphragms directly into the cores.

5.6 Design summary

Table 5.1 summarises the most appropriate design approaches for the ultimate limit state for the various elements:

Table 5.1 Design of elements for ultimate limit state

Element	Method	Section Reference	Comments
Floor slab	Manufacturers' data	5.2.1	Ensure relevant Nationally Determined Parameters (NDP) are adopted
Downstand non-composite beams	Capacity tables from design software[11]	5.2.2	Ensure relevant NDPs are adopted
Downstand composite beams	Design software	5.2.3	
Columns under axial load	Tabulated data	5.3.1	Ensure relevant NDPs are adopted
Columns under axial load and moments	Design software	5.3.2	
Vertical bracing	Design using tabulated data, taking account of local connection / eccentricity issues	5.4	
Horizontal bracing	Design using tabulated data, taking account of both local connection / eccentricity issues and connectivity to concrete	5.5	

6 ROBUSTNESS

6.1 Accidental design situations

In order to avoid the disproportionate collapse of buildings in the case of accidental situations, such as explosions, Section 2.1 of EN 1990 states two Principles and provides one Application Rule for the robustness of structures. These are as follows:

(4)P A structure shall be designed and executed in such a way that it will not be damaged by events such as:

- Explosion,
- Impact, and
- the consequences of human errors,

to an extent disproportionate to the original cause.

NOTE 1. The events to be taken into account are "those agreed for an individual project with the client and the relevant authority".

NOTE 2: Further information is given in EN 1991-1-7.

(5)P Potential damage shall be avoided or limited by appropriate choice of one or more of the following:

- avoiding, eliminating or reducing the hazards to which the structure can be subjected;.
- selecting a structural form which has low sensitivity to the hazards considered;
- selecting a structural form and design that can survive adequately the accidental removal or an individual member or a limited part of the structure, or the occurrence of acceptable localised damage;
- avoiding as far as possible structural systems that can collapse without warning;
- tying the structural members together.

(6) The basic requirements should be met:

- by the choice of suitable materials.
- by appropriate design and detailing, and
- by specifying control procedures for design, production, execution and use relevant to the particular project.

The strategy to be adopted with both identified and unidentified accidental actions is illustrated in Figure 6.1 and depends on three consequence classes that are set out in EN 1991-1-7[13] Appendix B.3 and discussed in Section 6.2.

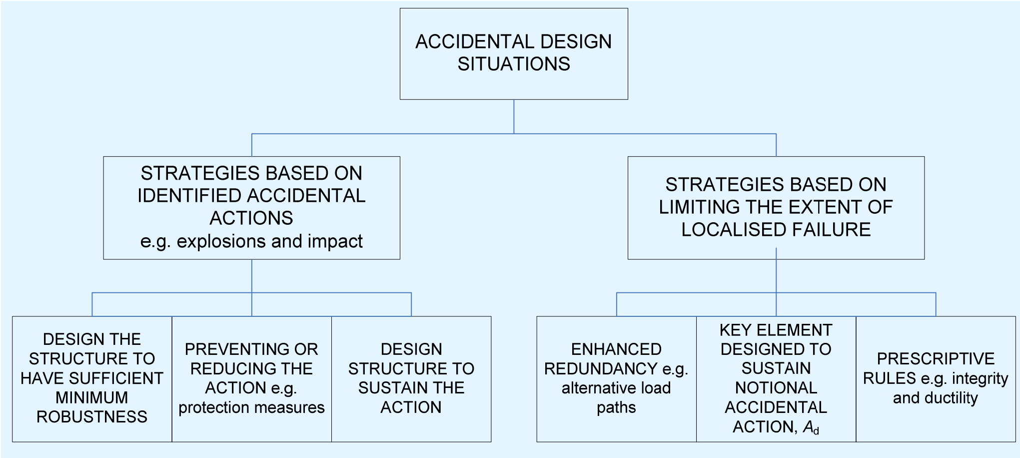


Figure 6.1 Strategies for accidental design situations

6.2 Consequence classes

As mentioned above, Appendix B.3 of EN 1990 defines three consequence classes:

CC1 Low consequences of failure
CC2 Medium consequences of failure
CC3 High consequences of failure

Class CC2 is subdivided by EN 1991-1-7[13] into CC2a (Lower risk group) and CC2b (Upper risk group). Medium rise buildings mostly fall within group CC2b, the criteria for which are reproduced Table 6.1.

Table 6.1 An example of building categorisation (taken from Table A.1 of EN 1991-1-7)

Consequence Class	Example of categorisation of building type and occupancy
CC2b Upper Risk Group	Hotels, flats, apartments and other residential buildings greater than 4 storeys but not exceeding 15 storeys Educational buildings greater than single storey but not exceeding 15 storeys Retailing premises greater than 3 storeys but not exceeding 15 storeys Offices greater than 4 storeys but not exceeding 15 storeys All buildings to which the public are admitted and which contain floor areas exceeding 2000 m^2 but not exceeding 5000 m^2 at each storey

EN 1991-1-7, § 3.2 and § 3.3 provide a wide range of possible general strategies for identified accidental actions and for limiting the extent of localised failure respectively. Apart from structures at specific risk from impact (EN 1991-1-7 Section 4) or internal explosion (EN 1991-1-7, § 5), this publication recommends that design of low and medium rise building structures in Consequence Class CC2b should generally involve the design for localised failure (see Section 6.3 of this document). The design of columns as key elements (see Section 6.4) is only appropriate where not all columns are continuous through to the basement; for example where they are supported by a transfer structure.

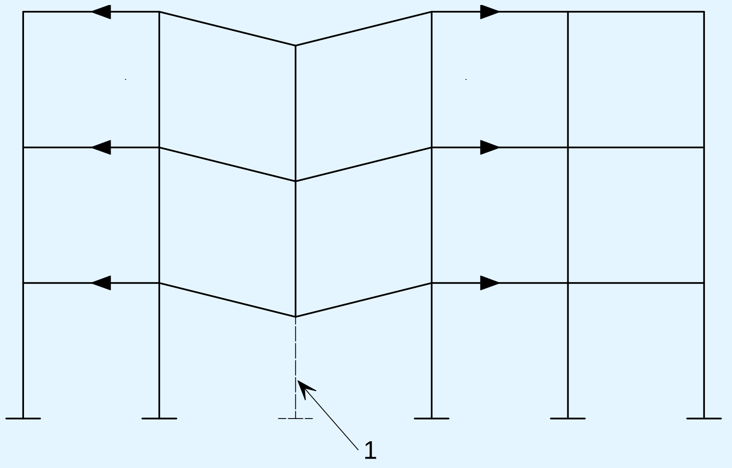
6.3 Design for the consequences of localised failure in multi-storey buildings

6.3.1 Design strategy

In multi-storey buildings, the requirement for robustness generally leads to a design strategy where the columns are tied into the rest of the structure. This should mean that any one length of column cannot easily be removed. However, should a length be removed by an accidental action, the floor systems should be able to develop catenary action, to limit the extent of the failure. This is illustrated diagrammatically in Figure 6.2. The recommendations in EN 1991-1-7[13], Annex A in relation to horizontal tying actions and vertical tying actions are related to this form of partial collapse.

Annex A does not prescribe a complete design model for this form of partial collapse – for example, the reaction to the horizontal forces in Figure 6.2 is not addressed. The rules in the Annex are best considered as prescriptive rules intended to produce structures that perform adequately in extreme circumstances and are not meant to be fully described systems of structural mechanics. The illogical practice of designing certain connections for considerable force, yet not making provision to transfer the forces any further, illustrates this point.

It is important to note that the requirements are not intended to ensure that the structure is still serviceable following some extreme event, but that damage is limited and that progressive collapse is prevented.

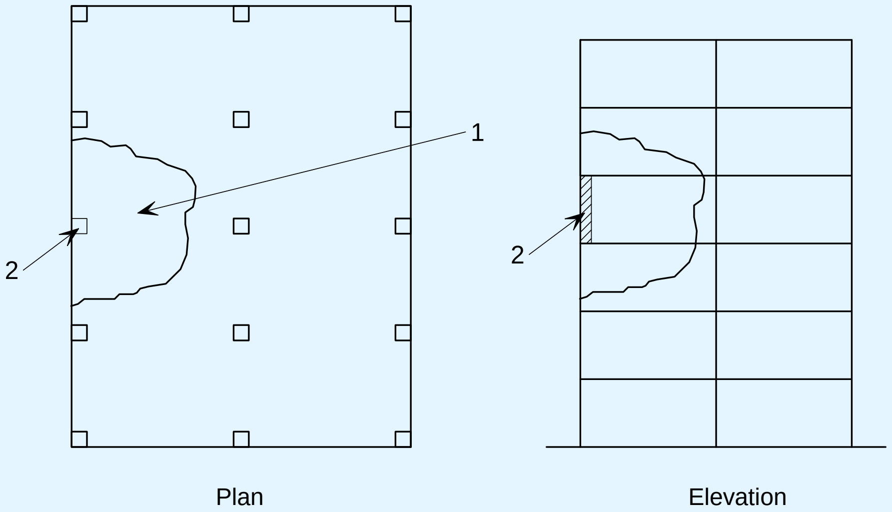


1 Column removed

Figure 6.2 Concept of robustness rules

6.3.2 Limit of admissible damage

The limit of admissible damage recommended in EN 1991-1-7, Annex A is shown in Figure 6.3.



1 Local damage not exceeding 15% of the floor area or 100 m^2, whichever is the smaller, in each of two adjacent storeys
2 Notional column to be removed

Figure 6.3 Recommended limit of admissible damage (taken from Figure A.1 of EN 1991-1-7)

6.3.3 Horizontal tying

EN 1991-1-7 § A.5 provides guidance on the horizontal tying of framed structures. It gives expressions for the design tensile resistance required for internal and perimeter ties.

For internal ties:

$$T_i = 0.8\ (g_k + \psi q_k)\ s\ L \text{ or } 75 \text{ kN, whichever is the greater} \qquad (A.1)$$

For perimeter ties:

$$T_p = 0.4\ (g_k + \psi q_k)\ s\ L \text{ or } 75 \text{ kN, whichever is the greater} \qquad (A.2)$$

where:

s is the spacing of ties

L is the span of the tie

ψ is the relevant factor in the expression for combination of action effects for the accidental design situation (i.e. ψ_1 or ψ_2 in accordance with expression (6.11b) of EN 1990[5]).The relevant National Annex should give further guidance on the ψ values to be adopted.

Note that tying forces do not necessarily need to be carried by the steelwork frame. A composite concrete floor, for example, can be used to tie columns together but must be designed to perform this function. Additional reinforcement may be required and a column (particularly an edge column) may need careful detailing to ensure the tying force is transferred between columns and slab. Reinforcing bars around a column, or threaded bars bolted into the steel column itself, have been successfully used.

If the tying forces are to be carried by the structural steelwork alone, the verification of tying resistance is entirely separate from that for resistance to vertical forces. The shear forces and tying forces are never applied at the same time. Furthermore, the usual requirement that members and connections remain serviceable under design loading is ignored when calculating resistance to tying, as ‘substantial permanent deformation of members and their connections is acceptable’. Guidance on the tying resistance of standard simple connections is presented in *Multi-storey steel buildings. Part 5: Joint design*[14].

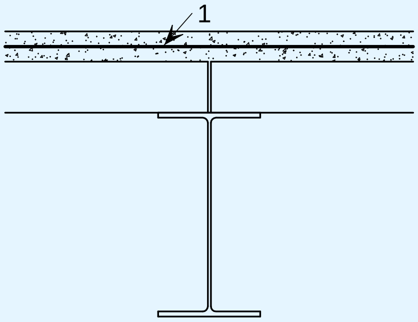
Frequently, ties may be discontinuous, or have no ‘anchor’ at the end distant to the column. The connection is simply designed for the applied force. This situation is also common at external columns, where only the local design of the connection is considered. The column itself is not designed to resist the tying force.

6.3.4 Tying of precast concrete floor units

EN 1991-1-7 § A.5.1(2) requires that when concrete or other heavy floor units are used (as floors), they should be tied in the direction of their span. The intention is to prevent floor units or floor slabs simply falling through the steel frame, if the steelwork is moved or removed due to some major trauma. Slabs must be tied to each other over supports and tied to edge beams. Tying forces may be determined from EN 1992-1-1[7], § 9.10.2 and the relevant National Annex.

Tying across internal supports

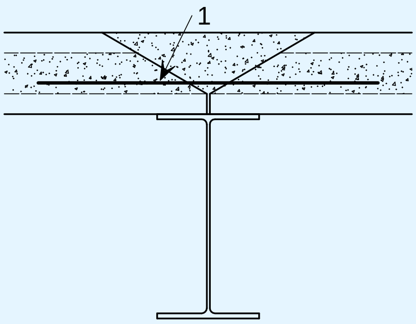
If the precast units have a structural screed, it may be possible to use the reinforcement in the screed to carry the tie forces, as shown in Figure 6.4, or to provide additional reinforcing bars.



1 Reinforcement in screed

Figure 6.4 Screed with reinforcement

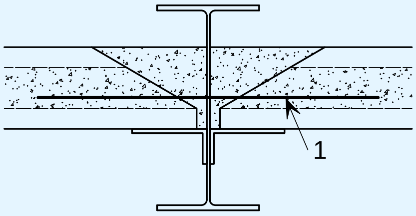
Alternatively, it may be possible to expose the voids in the precast planks and place reinforcing bars between the two units prior to concreting, as shown in Figure 6.5.



1 Reinforcement in core with concrete infill

Figure 6.5 Ties between hollow precast units

Special measures may be needed where precast planks are placed on shelf angles, as shown in Figure 6.6. When it is not possible to use reinforcement in the screed, straight reinforcement bars tying the precast units together are usually detailed to pass through holes drilled in the steel beam.

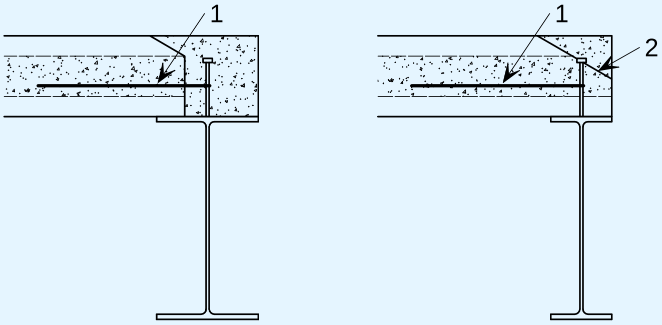


1 Reinforcing bar

Figure 6.6 Precast units on shelf angles

Tying to edge beams

Anchorage is best accomplished by exposing the voids in the plank and placing U-shaped bars around studs welded to the steelwork, as shown in Figure 6.7. In this Figure, the studs have been provided in order to achieve adequate anchorage; not for composite design of the edge beam. Figure 6.7b is a more complicated solution involving castellation of the plank edge (often on site) so that the plank fits around the stud and similar U-bars located in the voids prior to concreting. However, this does allow a narrower steel beam to be used.



a) b)

1 U-bar
2 Plank castellated around shear studs

Figure 6.7 Tying of precast planks to edge beams
Minimum flange width of: (a) 180 mm, (b) 120 mm

In some circumstances, the floor units cantilever past the edge beam. Tying in these situations is not straightforward, and a solution must be developed in collaboration with the frame supplier and floor unit manufacturer.

6.3.5 Vertical tying

EN 1991-1-7, A.6 provides guidance on the vertical tying of framed structures. It recommends that column splices should be capable of carrying an axial tension equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey. It does not specify which storey but it would be appropriate to use the largest value over the length down to the next splice, or to the base, if that is nearer).

In practice, this is not an onerous obligation and most splices designed for adequate stiffness and robustness during erection are likely to be sufficient to carry the axial tying force. Standardised splices are covered in *Multi-storey steel buildings. Part 5: Joint design*[14].

6.4 Key elements

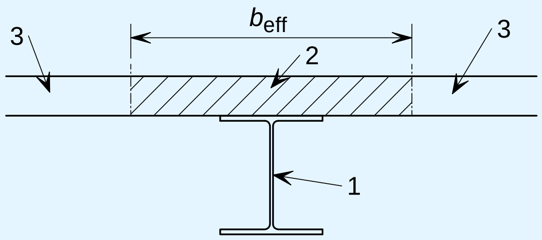
EN 1991-1-7[13], § A.8 provides guidance on the design of "key elements". It recommends that a key element should be capable of sustaining an accidental design action of A_{d} applied in horizontal and vertical directions (in one direction at a time) to the member and any attached components. The recommended value of A_{d} for building structures is 34 kN/m^2 applied to the surface area of the element in the most onerous direction. Any other structural component that provides "lateral restraint vital to the stability" of a key element should also be designed as a key element. Equation 6.11b of EN 1990[5] defines the combination of actions which needs to be considered.

When considering the accidental loading on a large area (e.g. on a floor slab supported by a transfer beam), it is reasonable to limit the area that is subjected to the 34 kN/m^2 load because a blast pressure is unlikely to be this high on all the surfaces of a large enclosed space.

The maximum area to be considered is not defined but could be inferred from the length of load-bearing wall to be considered (see EN 1991-1-7, § A.7) which is 2,25 times the storey height, say $2{,}25 \times 2{,}9 = 6{,}5$ m. Therefore, a maximum area that would be subjected to the 34 kN/m^2 load could be a $6{,}5 \times 6{,}5$ m square.

For the design of a key element, it is necessary to consider what components, or proportion of components, will remain attached to the element in the event of an incident. The application of engineering judgement will play a major part in this process. For framed construction, the walls and cladding will normally be non-structural. Therefore, it is likely that the majority of these will become detached from the key element during an incident, as shown in Figure 6.8.

For the column member key element shown in Figure 6.8, an accidental load of 34 kN/m^2 should be applied over a width b_{eff} for accidental loading about the major axis. The column section should be verified for the combination of moments and axial force using the design case given above. The accidental loading about the minor axis over a width of h (in this case) also needs to be considered. The accidental loading should only be considered as acting in one direction at a time and there is no requirement to consider a diagonal loading case, i.e. at an angle to the major and minor axes.



1 Key element
2 Portion of component that remains attached to key element after an incident
3 portions of component that are detached from key element during an incident

Figure 6.8 Component attached to a key element (column on plan)

Determining the width b_{eff} is very subjective. An estimate of what will remain attached to the key element (during a loading of 34 kN/m^2) will obviously depend on what is attached and how it is fixed to the element.

6.5 Risk assessment

Buildings which fall into consequence class 3 have to be assessed using risk assessment techniques. Annex B of EN 1991-1-7[13] provides information on risk assessment and B.9 provides guidance specific to buildings.

6.6 Design summary

- Determine the relevant consequence class from Appendix B.3 of EN 1990[5] (Section 6.2)
- Design members and connections to limit localised failure wherever possible. Columns will need to be designed as key elements where they are not continuous through to the basement; for example, where they finish on a transfer structure.
- For design for localised failure, adopt the design strategy, limit of admissible damage and horizontal and vertical tying rules described in Section 6.3.
- Where key elements have to be protected, the approaches outlined in Section 6.4 will have to be adopted.

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APPENDIX A WORKED EXAMPLES

1. Simply supported, laterally unrestrained beam

1.1. Scope

This example covers an I-section rolled profile beam, subject to bending about the major axis and restrained laterally at the supports only. The example includes:

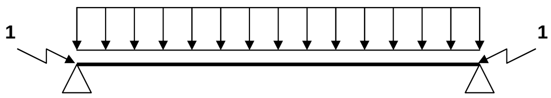
- the classification of the cross-section
- the calculation of bending resistance, including the exact calculation of the elastic critical moment for lateral-torsional buckling
- the calculation of shear resistance
- the calculation of the deflection at serviceability limit state.

This example does not include any shear buckling verification of the web.

1.2. Loading

The uniformly distributed loading comprises:

- self-weight of the beam,
- concrete slab,
- imposed load.



1 Lateral restraint

Figure A.1 Beam employed in this example, showing the lateral restraints

1.3. Partial safety factors

EN 1990

γ_G = 1,35 (permanent actions)

γ_Q = 1,50 (variable actions)

EN 1993-1-1 § 6.1 (1)

γ_{M0} = 1,0

γ_{M1} = 1,0

1.4. Basic data

Span length: 5,70 m

Bay width: 2,50 m

Slab depth: 120 mm

Partitions: 0,75 kN/m^2

Imposed load: 2,50 kN/m^2

Concrete density: 24 kN/m^3

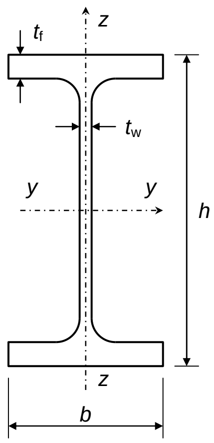
Steel grade: S235

Weight of the slab: $0,12 \times 24$ kN/m^3 = 2,88 kN/m^2

1.5. Choose a section

Try IPE 330 – Steel grade S235

1.5.1. Geometric properties



Depth	h	= 330 mm
Width	b	= 160 mm
Web thickness	t_w	= 7,5 mm
Flange thickness	t_f	= 11,5 mm
Root radius	r	= 18 mm
Mass		49,1 kg/m
Section area	A	= 62,6 cm^2

Second moment of area about mayor axis: I_y = 11770 cm^4

Second moment of area about minor axis: I_z = 788,1 cm^4

Torsional constant I_t = 28,15 cm^4

Warping constant I_w = 199100 cm^6

Elastic modulus about major axis: $W_{el,y}$ = 713,1 cm^3

Plastic modulus about major axis: $W_{pl,y}$ = 804,3 cm^3

Yield strength

EN 1993-1-1 Table 3.1

Steel grade: S235

The maximum thickness is 11,5 mm < 40 mm, so: f_y = 235 N/mm^2

Note: The National Annex may impose either the values of f_y from Table 3.1 or the values from the product standard.

1.5.2. Actions on the beam

Self weight of the beam : $(49,1 \times 9,81) \times 10^{-3}$ = 0,482 kN/m

Permanent action:

$G_k = 0{,}482 + (2{,}88 + 0{,}75) \times 2{,}50 = 9{,}56$ kN/m

Variable action (Imposed load):

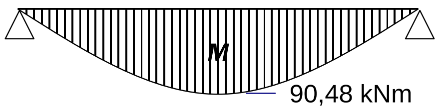
$Q_k = 2{,}5 \times 2{,}5 = 6{,}25$ kN/m

1.5.3. Effects on the beam

ULS combination: EN 1990 § 6.4.3.2

$\gamma_G\, G_k + \gamma_Q\, Q_k = 1{,}35 \times 9{,}56 + 1{,}50 \times 6{,}25 = 22{,}28$ kN/m

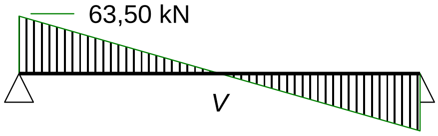
Bending moment diagram



Maximum bending moment occurs at mid span and is given by:

$$M_{y,Ed} = \frac{wL^2}{8} = \frac{22{,}28 \times 5{,}70^2}{8} = 90{,}48 \text{ kNm}$$

Shear force diagram



Maximum shear force at supports:

$V_{Ed} = 0{,}5 \times 22{,}28 \times 5{,}70 = 63{,}50$ kN

1.5.4. Section classification :

The parameter ε is derived from the yield strength: EN 1993-1-1 Table 5.2

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{235}} = 1{,}0$$

Outstand flange: flange under uniform compression

$c = (b - t_w - 2\,r) / 2 = (160 - 7{,}5 - 2 \times 18) / 2 = 58{,}25$ mm

$c/t_f = 58{,}25 / 11{,}5 = 5{,}07 \leq 9\,\varepsilon = 9{,}0$ Flange Class 1

Internal compression part: web under pure bending

$c = h - 2\,t_f - 2\,r = 330 - 2 \times 11{,}5 - 2 \times 18 = 271$ mm

$c / t_w = 271 / 7{,}5 = 36{,}1 < 72\,\varepsilon = 72$ Web Class 1

The class of the cross-section is the highest class (i.e. the least favourable) of the flange and the web. Therefore, the overall section is Class 1.

For Class 1 sections, the ULS verifications must be based on the plastic resistance of the cross-section.

1.5.5. Section resistance

Cross-sectional moment resistance

The design cross-sectional resistance is: EN 1993-1-1 § 6.2.5

$M_{c,Rd} = W_{pl,y} f_y / \gamma_{M0} = (804{,}3 \times 235 / 1{,}0) \times 10^{-3} = 189{,}01$ kNm

The section must verify that $M_{y,Ed} / M_{c,Rd} < 1{,}0$

$M_{y,Ed} / M_{c,Rd} = 90{,}48 / 189{,}01 = 0{,}479 < 1{,}0$ OK

Lateral-torsional buckling resistance

To determine the design lateral-torsional buckling resistance, the reduction factor for lateral-torsional buckling must be determined. The following calculation determines this factor by means of the elastic critical moment.

Elastic critical moment

The critical moment may be calculated from the following expression:

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(k L)^2} \left\{ \sqrt{\left(\frac{k}{k_w}\right)^2 \frac{I_w}{I_z} + \frac{(k L)^2 G I_t}{\pi^2 E I_z} + (C_2 z_g)^2} - C_2 z_g \right\}$$

SN003[4]

where:

E	is Young's modulus:	$E = 210000$ N/mm^2
G	is the shear modulus:	$G = 80770$ N/mm^2
L	is the span:	$L = 5{,}70$ m

In the expression for M_{cr}, the following simplifications can be made:

$k = 1$ since the compression flange is free to rotate about the weak axis of the cross-section,

$k_w = 1$ since warping is not prevented at the ends of the beam.

z_g is the distance from the loading point to the shear centre:

$z_g = h / 2 = +165$ mm

(z_g is positive when the loads act towards the shear centre)

The C_1 and C_2 coefficients depend on the bending moment diagram. For a uniformly distributed load and $k = 1$: SN003[4]

$C_1 = 1{,}127$

$C_2 = 0{,}454$

Therefore:

$$\frac{\pi^2 E I_z}{(k L)^2} = \frac{\pi^2 \times 210000 \times 788{,}1 \times 10^4}{(5700)^2} \times 10^{-3} = 502{,}75 \text{ kN}$$

$$C_2 z_g = 0{,}454 \times 165 = +74{,}91 \text{ mm}$$

M_{cr} =

$$1{,}127 \times 502{,}75 \times \left\{ \sqrt{\frac{199100}{788{,}1} \times 100 + \frac{80770 \times 281500}{502750} + (74{,}91)^2} - 74{,}91 \right\} \times 10^{-3}$$

M_{cr} = 113,9 kNm

Non-dimensional slenderness

The non-dimensional slenderness is obtained from:

$$\bar{\lambda}_{\mathrm{LT}} = \sqrt{\frac{W_{\mathrm{pl,y}}\, f_{\mathrm{y}}}{M_{\mathrm{cr}}}} = \sqrt{\frac{804300 \times 235 \times 10^{-6}}{113{,}9}} = 1{,}288$$

EN 1993-1-1 § 6.3.2.2 (1)

For rolled profiles: $\bar{\lambda}_{\mathrm{LT,0}} = 0{,}4$

EN 1993-1-1 § 6.3.2.3(1)

Note: the value of $\bar{\lambda}_{\mathrm{LT,0}}$ may be given in the National Annex. The recommended value is 0,4.

So $\bar{\lambda}_{\mathrm{LT}} = 1{,}288 \; > \; \bar{\lambda}_{\mathrm{LT,0}}$

Reduction factor, χ_{LT}

EN 1993-1-1 § 6.3.2.3 (1)

For rolled sections, the reduction factor for lateral-torsional buckling is calculated by:

$$\chi_{\mathrm{LT}} = \frac{1}{\phi_{\mathrm{LT}} + \sqrt{\phi_{\mathrm{LT}}^2 - \beta\,\bar{\lambda}_{\mathrm{LT}}^2}} \quad \text{but} \quad \begin{cases} \chi_{\mathrm{LT}} \leq 1{,}0 \\ \chi_{\mathrm{LT}} \leq \dfrac{1}{\bar{\lambda}_{\mathrm{LT}}^2} \end{cases}$$

where:

$$\phi_{\mathrm{LT}} = 0{,}5 \left[1 + \alpha_{\mathrm{LT}} \left(\bar{\lambda}_{\mathrm{LT}} - \bar{\lambda}_{\mathrm{LT,0}} \right) + \beta\,\bar{\lambda}_{\mathrm{LT}}^2 \right]$$

α_{LT} is the imperfection factor for LTB. When applying the method for rolled profiles, the LTB curve has to be selected from Table 6.5:

EN 1993-1-1 Table 6.5 Table 6.3

For h/b = 330 / 160 = 2,06 > 2

Therefore use curve ‘c’ (α_{LT} = 0,49)

$\bar{\lambda}_{\mathrm{LT,0}} = 0{,}4$ and β = 0,75

Note: the values of $\bar{\lambda}_{\mathrm{LT,0}}$ and β may be given in the National Annex. The recommended values are 0,4 and 0,75 respectively.

$$\phi_{\mathrm{LT}} = 0{,}5 \left[1 + 0{,}49 \left(1{,}288 - 0{,}4 \right) + 0{,}75 \times (1{,}288)^2 \right] = 1{,}340$$

and:

$$\chi_{\mathrm{LT}} = \frac{1}{1{,}340 + \sqrt{(1{,}340)^2 - 0{,}75 \times (1{,}288)^2}} = 0{,}480$$

$\chi_{LT} = 0{,}480 \quad < \quad 1{,}0 \quad$ OK

and: $\chi_{LT} = 0{,}480 \quad < \quad 1 / \bar{\lambda}_{LT}^{\,2} = 0{,}603 \quad$ OK

The influence of the moment distribution on the design buckling resistance moment of the beam is taken into account through the *f*-factor: *EN 1993-1-1 § 6.3.2.3 (2)*

$$f = 1 - 0{,}5\left(1 - k_c\right)\left[1 - 2\left(\bar{\lambda}_{LT} - 0{,}8\right)^2\right] \quad \text{but} \leq 1{,}0$$

where: *EN 1993-1-1 Table 6.6*

$k_c \quad = 0{,}94$

$\therefore f \quad = 1 - 0{,}5\ (1 - 0{,}94)\ [1 - 2\ (1{,}288 - 0{,}8)^2] = 0{,}984$

$\therefore\ \chi_{LT,mod} = \chi_{LT} / f = 0{,}480 / 0{,}984 = 0{,}488$

Design buckling resistance moment

$M_{b,Rd} = \chi_{LT,mod}\, W_{pl,y}\, f_y / \gamma_{M1}$ *EN 1993-1-1 § 6.3.2.1*

$M_{b,Rd} = (0{,}488 \times 804300 \times 235 / 1{,}0) \times 10^{-6} = 92{,}24$ kNm

$M_{y,Ed} / M_{b,Rd} = 90{,}48 / 92{,}24 = 0{,}981 \quad < \quad 1{,}0 \quad$ OK

Shear Resistance

In the absence of torsion, the plastic shear resistance is directly related to the shear area, which is given by: *EN 1993-1-1 § 6.2.6 (3)*

$A_v \quad = A - 2\, b\, t_f + (t_w + 2\, r)\, t_f$

$A_v \quad = 6260 - 2 \times 160 \times 11{,}5 + (7{,}5 + 2 \times 18) \times 11{,}5 = 3080\ \text{mm}^2$

$$V_{pl,Rd} = \frac{A_v\ (f_y / \sqrt{3})}{\gamma_{M0}} = \frac{3080 \times (235 / \sqrt{3})}{1{,}0} = 417{,}9\ \text{kN}$$ *EN 1993-1-1 § 6.2.6 (2)*

$V_{Ed} / V_{pl,Rd} = 63{,}50 / 417{,}9 = 0{,}152 \quad < \quad 1{,}0 \qquad$ OK

Shear buckling need not be taken into account when:

$h_w / t_w \leq 72\ \varepsilon / \eta$ *EN 1993-1-1 § 6.2.6 (6)*

where:

$\eta \quad$ may be conservatively taken as 1,0

$h_w / t_w = (330 - 2 \times 11{,}5) / 7{,}5 = 40{,}9 \quad < \quad 72 \times 1 / 1{,}0 = 72$

Note: No interaction of moment and shear has to be considered since the maximum moment is obtained at mid-span and the maximum shear force is obtained at supports. Generally for combined bending and shear see EN 1993-1-1, § 6.2.8.

1.5.6. Serviceability Limit State verification

SLS Combination

EN 1990 § 6.5.3

$G_k + Q_k = 9{,}56 + 6{,}25 = 15{,}81$ kN/m

Deflection due to $G_k + Q_k$:

$$w = \frac{5\,(G+Q)\,L^4}{384\,E\,I_y} = \frac{5 \times 15{,}81 \times (5700)^4}{384 \times 210000 \times 11770 \times 10^4} = 8{,}8 \text{ mm}$$

The deflection under ($G_k + Q_k$) is $L/648$ OK

EN 1993-1-1 § 7.2.1

Note: 1 the deflection limits should be specified by the client. The National Annex may specify some limits.

Note 2: regarding vibrations, the National Annex may specify limits for the frequency.

EN 1993-1-1 § 7.2.3

2. Simply Supported Beam with Intermediate Lateral Restraints

2.1. Scope

This example deals with a simply supported roof beam, with intermediate lateral restraints, under a uniformly distributed load:

- self-weight of the beam
- roofing with purlins
- climatic loads.

The beam is a rolled I-profile in bending about the strong axis.

This example includes:

- the classification of the cross-section
- the calculation of bending resistance
- the calculation of shear resistance
- the calculation of the deflection at serviceability limit state.

2.2. Partial safety factor

EN 1990 Table A1.2(B)

$\gamma_{G,sup} = 1,35$ (permanent loads)

$\gamma_{G,inf} = 1,0$ (permanent loads)

$\gamma_{Q} = 1,50$ (variable loads)

EN 1993-1-1 § 6.1 (1)

$\gamma_{M0} = 1,0$

$\gamma_{M1} = 1,0$

2.3. Basic data

Span length: 15,00 m

Bay width: 6,00 m

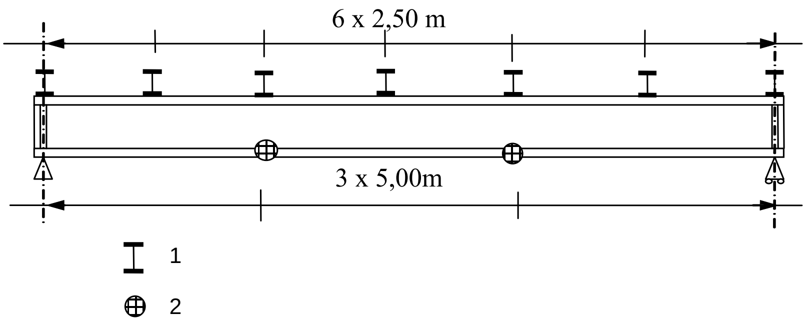
Roof: 0,30 kN/m^2

Climatic load: Snow 0,60 kN/m^2

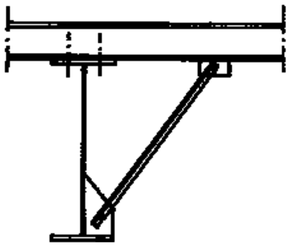
Climatic load: Wind - 0,50 kN/m^2 (suction)

Steel grade: S235

The climatic loads are characteristic values assumed to have been calculated according to EN 1991.



1 : Lateral restraints (purlins)
2 : Lateral restraints (bracing system)



Bracing system

The beam is laterally restrained at the supports. The upper flanges are restrained by the purlins (at 2,50 m spacing) and the lower flanges by struts (at 5,00 m spacing).

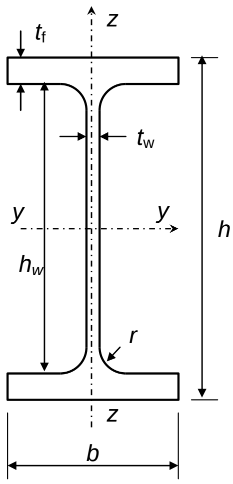
The beam is fabricated with a pre-camber equal to l/500, w_c = 30 mm.

2.4. Choose a section

Try IPE 400 – Steel grade S235

2.4.1. Geometric properties

Depth	h	= 400 mm
Web Depth	h_w	= 373 mm
Width	b	= 180 mm
Web thickness	t_w	= 8,6 mm
Flange thickness	t_f	= 13,5 mm
Root radius	r	= 21 mm
Mass		66,3 kg/m
Section area	A	= 84,46 cm^2



Second moment of area about the major axis I_y $= 23130\ \text{cm}^4$

Second moment of area about the minor axis I_z $= 1318\ \text{cm}^4$

Torsion constant I_t $= 51{,}08\ \text{cm}^4$

Warping constant I_w $= 490\,000\ \text{cm}^6$

Elastic section modulus about the major axis $W_{el,y}$ $= 1156\ \text{cm}^3$

Plastic section modulus about the major axis $W_{pl,y}$ $= 1307\ \text{cm}^3$

Yield strength

Steel grade S235

The maximum thickness is 13,5 mm < 40 mm, so: $f_y = 235\ \text{N/mm}^2$

EN 1993-1-1
Table 3.1

2.5. ULS verification

2.5.1. Actions on the beam

Self weight of the beam: $(66{,}3 \times 9{,}81) \times 10^{-3} = 0{,}65$ kN/m

Permanent load:

$G_k = 0{,}65 + 0{,}30 \times 6{,}00 = 2{,}45$ kN/m

Climatic load:

$Q_s = 0{,}60 \times 6{,}0 = 3{,}60$ kN/m

$Q_w = -0{,}50 \times 6{,}0 = -3{,}00$ kN/m

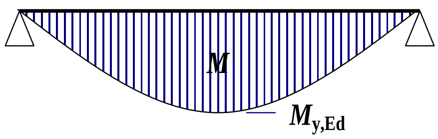
2.5.2. Effects on the beam due to ULS combinations:

EN 1990
§ 6.4.3.2
Expression 6.10

Combination 1 $\gamma_{G,sup}\, G_k + \gamma_Q\, Q_s = 1{,}35 \times 2{,}45 + 1{,}50 \times 3{,}60 = 8{,}71$ kN/m

Combination 2 $\gamma_{G,inf}\, G_k + \gamma_Q\, Q_w = 1{,}00 \times 2{,}45 - 1{,}50 \times 3{,}00 = -2{,}05$ kN/m

Bending moment diagram

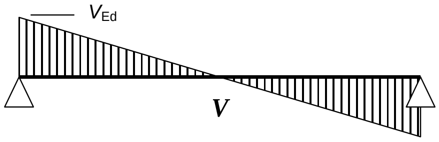


Maximum moment at mid span:

Combination 1 $M_{y,Ed} = \dfrac{wL^2}{8} = \dfrac{8{,}71 \times 15^2}{8} = 244{,}97$ kNm

Combination 2 $M_{y,Ed} = \dfrac{wL^2}{8} = \dfrac{-2{,}05 \times 15^2}{8} = -57{,}66$ kNm

Shear force diagram



Maximum shear force at supports :

Combination 1 $V_{Ed} = 0,5 \times 8,71 \times 15 = 65,33$ kN

Combination 2 $V_{Ed} = 0,5 \times -2,05 \times 15 = -15,38$ kN

2.5.3. Section classification

The parameter ε is derived from the yield strength:

EN 1993-1-1 Table 5.2

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{235}} = 1,0$$

Outstand flange: flange under uniform compression

$c = (b - t_w - 2\,r) / 2 = (180 - 8,6 - 2 \times 21)/2 = 64,7$ mm

$c/t_f = 64,7 / 13,5 = 4,79 \leq 9\,\varepsilon = 9$ Flange class 1

Internal compression part: web under pure bending

$c = h - 2\,t_f - 2\,r = 400 - 2 \times 13,5 - 2 \times 21 = 331$ mm

$c / t_w = 331 / 8,6 = 38,49 < 72\,\varepsilon = 72$ Web class 1

The class of the cross-section is the highest class (i.e. the least favourable) between the flange and the web. Therefore, the overall section is Class 1.

EN 1993-1-1 Table 5.2

For Class 1 sections, the ULS verifications should be based on the plastic resistance of the cross-section.

2.5.4. Section resistance

Cross-sectional moment resistance

The design cross-sectional resistance is:

EN 1993-1-1 § 6.2.5

$M_{c,Rd} = W_{pl,y}\, f_y / \gamma_{M0} = (1307 \times 235 / 1,0) \times 10^{-3} = M_{c.Rd} = 307,15$ kNm

Combination 1 $M_{y,Ed} / M_{c,Rd} = 244,97 / 307,15 = 0,798 < 1,0$ OK

Combination 2 $M_{y,Ed} / M_{c,Rd} = 57,66 / 307,15 = 0,188 < 1,0$ OK

Lateral-torsional buckling verification using the simplified assessment methods for beams with restraints in buildings:

In buildings, members with discrete lateral restraint to the compression flange are not susceptible to lateral-torsional buckling if the length L_c between restraints or the resulting equivalent compression flange slenderness $\bar{\lambda}_f$ satisfies:

EN 1993-1-1 § 6.3.2.4 (1)B

$$\bar{\lambda}_{\rm f} = \frac{k_{\rm c} L_{\rm c}}{i_{\rm f,z}\lambda_1} \leq \bar{\lambda}_{\rm c0} \frac{M_{\rm c,Rd}}{M_{\rm y,Ed}}$$

where:

$M_{\rm y,Ed}$ is the maximum design value of the bending moment within the restraint spacing

$k_{\rm c}$ is a slenderness correction factor for moment distribution between restraints, see EN 1993-1-1, Table 6.6

$i_{\rm f,z}$ is the radius of gyration of the compression flange including 1/3 of the compressed part of the web area, about the minor axis of the section

$\bar{\lambda}_{\rm c0}$ is the slenderness parameter of the above compression element

$\bar{\lambda}_{\rm c0} = \bar{\lambda}_{\rm LT,0} + 0{,}10$

For rolled profiles, $\bar{\lambda}_{\rm LT,0} = 0{,}40$

Note: The slenderness limit $\bar{\lambda}_{\rm c0}$ may be given in the National Annex.

EN 1993-1-1 6.3.2.3 (1)

$$\lambda_1 = \pi \sqrt{\frac{E}{f_{\rm y}}} = 93{,}9\varepsilon \quad \text{and} \quad \varepsilon = \sqrt{\frac{235}{f_{\rm y}}} = \sqrt{\frac{235}{235}} = 1$$

$I_{\rm f,z} = [1318 - (2 \times 37{,}3 / 3) \times 0{,}86^3 / 12] / 2 = 658{,}34 \text{ cm}^4$

$A_{\rm f,z} = [84{,}46 - (2 \times 37{,}3 / 3) \times 0{,}86] / 2 = 31{,}54 \text{ cm}^2$

$$i_{\rm f,z} = \sqrt{\frac{658{,}34}{31{,}54}} = 4{,}57 \text{ cm}$$

$W_{\rm y} = W_{\rm pl,y} = 1307 \text{ cm}^3$

EN 1993-1-1 6.3.2.3

$$\lambda_1 = \pi \sqrt{\frac{E}{f_y}} = 93{,}9$$

$\bar{\lambda}_{\rm c0} = 0{,}40 + 0{,}10 = 0{,}50$

$$M_{\rm c,Rd} = W_{\rm y} \frac{f_{\rm y}}{\gamma_{\rm M1}} = \left(1307 \times \frac{235}{1{,}0}\right) \times 10^{-3} = 307{,}15 \text{ kNm}$$

Combination 1

EN 1993-1-1 Table 6.6

Note: Between restraints in the centre of the beam, where the moment is a maximum, the moment distribution can be considered as constant.

$k_{\rm c} = 1$

$L_{\rm c} = 2{,}50 \text{ m}$

$$\bar{\lambda}_{\rm f} = \frac{1 \times 250}{4{,}57 \times 93{,}9} = 0{,}583$$

$$\bar{\lambda}_{c0}\, M_{c,Rd} / M_{y,Ed} = 0,50 \times \frac{307,15}{244,97} = 0,627$$

$$\bar{\lambda}_{f} = 0,583 \leq \bar{\lambda}_{c0}\, M_{c,Rd} / M_{y,Ed} = 0,627 \quad \text{OK}$$

Combination 2

$k_c = 1$

$L_c = 5,00$ m

$$\bar{\lambda}_{f} = \frac{1 \times 500}{4,57 \times 93,9} = 1,165$$

$$\bar{\lambda}_{c0}\, M_{c,Rd} / M_{y,Ed} = 0,50 \times \frac{307,15}{57,66} = 2,663$$

$$\bar{\lambda}_{f} = 1,165 \leq \bar{\lambda}_{c0}\, M_{c,Rd} / M_{y,Ed} = 2,663 \quad \text{OK}$$

Shear Resistance

In the absence of torsion, the shear plastic resistance depends on the shear area, which is given by:

EN 1993-1-1 § 6.2.6 (3)

$A_v = A - 2\, b\, t_f + (t_w + 2\, r)\, t_f$

$A_v = 8446 - 2 \times 180 \times 13,5 + (8,6 + 2 \times 21) \times 13,5 = 4269 \text{ mm}^2$

EN 1993-1-1 § 6.2.6 (2)

$$V_{pl,Rd} = \frac{A_{v,z}\,(f_y/\sqrt{3})}{\gamma_{M0}} = \frac{4269 \times (235/\sqrt{3})}{1,0} \Big/ 1000 = 579,21 \text{ kN}$$

$V_{Ed} / V_{pl,Rd} = 65,33 / 579,21 = 0,113 < 1$ OK

EN 1993-1-1 § 6.2.6 (6)

Note that the verification to shear buckling is not required when :

$h_w / t_w \leq 72\, \varepsilon / \eta$

The value η may conservatively be taken as 1,0

$h_w / t_w = (400 - 2 \times 13,5) / 8,6 = 43,37 < 72 \times 1 / 1,0 = 72$

Note: No interaction between moment and shear has to be considered, since the maximum moment is obtained at mid-span and the maximum shear force is obtained at supports. Generally for combined bending and shear see EN 1993-1-1, § 6.2.8.

2.6. Serviceability Limit State verification

2.6.1. Actions on the beam

EN 1990 § 6.5.3 § A1.4.2

Characteristic combination:

$G_k + Q_s = 2,45 + 3,60 = 6,05$ kN/m

2.6.2. Deflection due to $G_k + Q_s$:

$$w_{tot} = \frac{5\,(G + Q_s)\,L^4}{384\,E\,I_y} = \frac{5 \times 6{,}05 \times (15000)^4}{384 \times 210000 \times 23130 \times 10^4} = 82{,}10 \text{ mm}$$

EN 1990 § A1.4.3

$w_c = 30$ mm pre-camber

$w_{max} = w_{tot} - w_c = 82{,}10 - 30 = 52{,}10$ mm

The deflection under ($G_k + Q_s$) is $L/288$.

2.6.3. Deflection due to Q_s :

$$w_3 = \frac{5\,(Q_s)\,L^4}{384\,E\,I_y} = \frac{5 \times 3{,}60 \times (15000)^4}{384 \times 210000 \times 23130 \times 10^4} = 48{,}90 \text{ mm}$$

The deflection under Q_s is $L/307$.

Note: The limits of deflection should be specified by the client. The National Annex may specify some limits.

EN 1993-1-1 § 7.2.1

3. Simply Supported, Secondary Composite Beam

3.1. Scope

This example covers the design of a composite floor beam of a multi-storey building according to the data given below. The beam is assumed to be fully propped during construction.

3.2. Loading

The following distributed loads are applied to the beam:

- self-weight of the beam
- concrete slab
- imposed load.

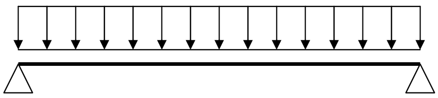


Figure A.2 Composite secondary beam to be designed in this example

The beam is a rolled I profile in bending about the strong axis. This example includes:

- the classification of the cross-section
- the calculation of the effective width of the concrete flange
- the calculation of shear resistance of a headed stud
- the calculation of the degree of shear connection
- the calculation of bending resistance
- the calculation of shear resistance
- the calculation of longitudinal shear resistance of the slab
- the calculation of deflection at serviceability limit state.

This example does not include any shear buckling verification of the web.

3.3. Partial factors

γ_G = 1,35 (permanent loads)

γ_Q = 1,50 (variable loads)

EN 1990

γ_{M0} = 1,0

γ_{M1} = 1,0

EN 1993-1-1 § 6.1 (1)

γ_V = 1,25

EN 1994-1-1 § 6.6.3.1

γ_C = 1,5

EN 1992-1-1

3.4. Basic data

The profiled steel sheeting is transverse to the beam.

Span length: 7,50 m

Bay width: 3,00 m

Slab depth: 12 cm

Partitions: 0,75 kN/m^2

Imposed load: 2,50 kN/m^2

Reinforced Concrete density: 25 kN/m^3

Steel grade: S355

3.5. Choose section

Try IPE 270

3.5.1. Geometric properties

Depth h_a = 270 mm

Width b = 135 mm

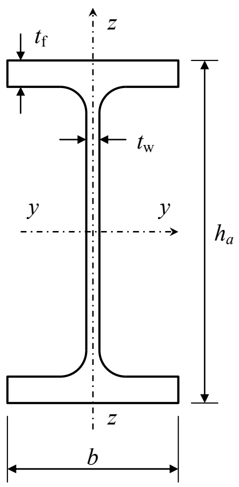
Web thickness t_w = 6,6 mm

Flange thickness t_f = 10,2 mm

Root radius r = 15 mm

Mass 36,1 kg/m

Section area A_a = 45,95 cm^2



Second moment of area about the major axis I_y = 5790 cm^4

Elastic modulus about the major axis $W_{el,y}$ = 428,9 cm^3

Plastic modulus about the major axis $W_{pl.y}$ = 484,0 cm^3

Modulus of elasticity of steel E_a = 210000 N/mm^2

Yield strength

Steel grade S355

The maximum thickness is 10,2 mm < 40 mm, so: $f_y = 355$ N/mm^2

EN 1993-1-1 Table 3.1

Note: The National Annex may impose either the values of f_y from Table 3.1 or the values from the product standard.

Profiled steel sheeting:

Thickness of sheet	t	= 0,75 mm
Slab depth	h	= 120 mm
Overall depth of the profiled steel sheeting	h_p	= 58 mm
Trapezoidal ribs	b_1	= 62 mm
	b_2	= 101 mm
	e	= 207 mm

Connectors:

Diameter	d	= 19 mm
Overall nominal height	h_{sc}	= 100 mm
Ultimate tensile strength	f_u	= 450 N/mm^2
Number of shear connectors studs	n	$= L / e = 7500 / 207 = 36$
Number of studs per rib	n_r	= 1

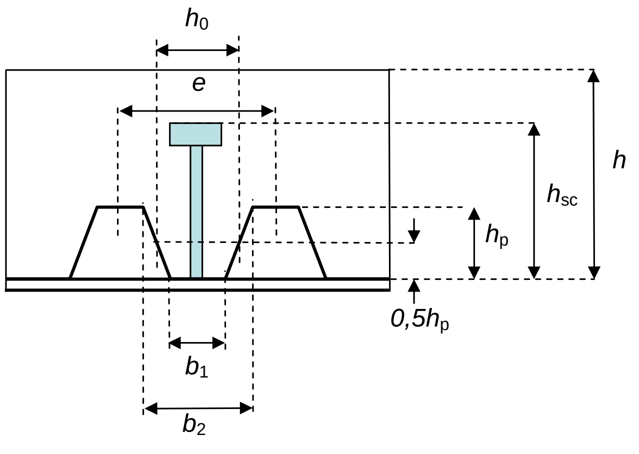


Figure A.3 Trapezoidal decking

Concrete parameters: C 25/30

Value of the compressive strength at 28 days $f_{ck} = 25$ N/mm^2

Modulus of elasticity of concrete $E_{cm} = 33000$ N/mm^2

EN 1992-1-1 § 3.1.3 Table 3.1

To take into account the troughs of the profiled steel sheeting, the weight of the slab is taken as:

$$25 \times 3{,}0 \times \left(0{,}12 - 5 \times \frac{0{,}101 + 0{,}062}{2} \times 0{,}058\right) = 7{,}2 \text{ kN/m}$$

Self weight of the beam: $(36{,}1 \times 9{,}81) \times 10^{-3} = 0{,}354$ kN/m

Permanent load:

$G_k = 0{,}354 + 7{,}2 + 0{,}75 \times 3{,}0 = 9{,}80$ kN/m

Variable load (Imposed load):

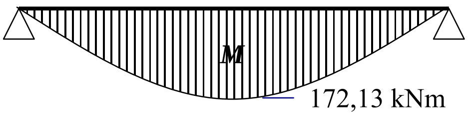
$Q_k = 2{,}5 \times 3{,}0 = 7{,}50$ kN/m

3.6. ULS Combination:

EN 1990
§ 6.4.3.2

$\gamma_G\, G_k + \gamma_Q\, Q_k = 1{,}35 \times 9{,}80 + 1{,}50 \times 7{,}50 = 24{,}48$ kN/m

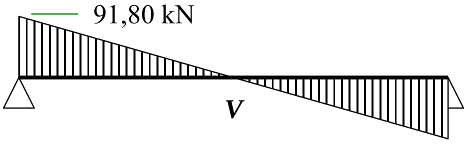
Bending moment diagram



Maximum moment at mid span :

$M_{y,Ed} = 0{,}125 \times 24{,}48 \times 7{,}50^2 = 172{,}13$ kNm

Shear force diagram



Maximum shear force at supports:

$V_{Ed} = 0{,}5 \times 24{,}48 \times 7{,}50 = 91{,}80$ kN

Section classification:

EN 1993-1-1
Table 5.2

The parameter ε is derived from the yield strength:

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0{,}81$$

Note: The classification is carried out for the non composite beam. For the composite beam, the classification is more favourable.

3.6.1. Section classification

EN 1993-1-1
Table 5.2

Outstand flange: flange under uniform compression

$c = (b - t_w - 2\, r) / 2 = (135 - 6{,}6 - 2 \times 15)/2 = 49{,}2$ mm

$c/t_f = 49{,}2 / 10{,}2 = 4{,}82 \;\; \leq \;\; 9\, \varepsilon = 7{,}29$ Flange Class 1

Internal compression part

$c = h - 2\,t_f - 2\,r = 270 - 2 \times 10{,}2 - 2 \times 15 = 219{,}6$ mm

$c / t_w = 219{,}6 / 6{,}6 = 33{,}3 \quad < \quad 72\,\varepsilon = 58{,}3$ Web Class 1

The class of the cross-section is the highest class (i.e. the least favourable) between the flange and the web.

So the ULS verifications should be based on the plastic resistance of the cross-section since the Class is 1.

3.6.2. Effective width of concrete flange

At mid-span, the total effective width may be determined by: *EN 1994-1-1 Figure 5.1*

$$b_{\text{eff,1}} = b_0 + \sum b_{\text{ei}}$$

b_0 is the distance between the centres of the outstand shear connectors, in this case $b_0 = 0$

b_{ei} is the value of the effective width of the concrete flange on each side of the web, $b_{\text{ei}} = L_e / 8$ but $\leq b_i = 3{,}0$ m

$b_{\text{eff,1}} = 0 + 7{,}5 / 8 = 0{,}9375$ m

$\therefore\ b_{\text{eff}} = 2 \times 0{,}9375 = 1{,}875 \text{ m} < 3{,}0$ m

At the ends, the total effective width is determined by: *EN 1994-1-1 Figure 5.1*

$$b_{\text{eff,0}} = b_0 + \sum \beta_i b_{\text{ei}}$$

where:

$\beta_i = (0{,}55 + 0{,}025\, L_e / b_{\text{ei}})$ but $\leq 1{,}0$

$= (0{,}55 + 0{,}025 \times 7{,}5 / 0{,}9375) = 0{,}75$

$b_{\text{eff,0}} = 0 + 0{,}75 \times 7{,}5 / 8 = 0{,}703$ m

$\therefore\ b_{\text{eff}} = 2 \times 0{,}703 = 1{,}406 \text{ m} < 3{,}0$ m

3.6.3. Design shear resistance of a headed stud

The shear resistance of each stud may be determined by:

$$P_{\text{Rd}} = k_t \times \text{Min}\left(\frac{0{,}8 f_u \pi d^2 / 4}{\gamma_V} ; \frac{0{,}29 \alpha d^2 \sqrt{f_{\text{ck}} E_{\text{cm}}}}{\gamma_V} \right)$$

EN 1994-1-1 § 6.6.3.1

$h_{\text{sc}} / d = 100 / 19 = 5{,}26 > 4$, so $\alpha = 1$

Reduction factor (k_t)

For sheeting with ribs transverse to the supporting beam, the reduction factor for shear resistance is calculated by: *EN 1994-1-1 § 6.6.4.2 Table 6.2*

$$k_t = \frac{0{,}7}{\sqrt{n_r}} \frac{b_0}{h_p} \left(\frac{h_{\text{sc}}}{h_p} - 1 \right)$$ but $\leq k_{\text{tmax}}$ for profiled sheeting with holes.

where:

n_r = 1

h_p = 58 mm

b_0 = 82 mm

h_{sc} = 100 mm

$$\therefore\ k_t = \frac{0,7}{\sqrt{1}}\frac{82}{58}\left(\frac{100}{58}-1\right) = 0,717\ \leq k_{tmax} = 0,75$$

$$P_{Rd} = 0,717 \times \mathrm{Min}\left(\frac{0,8 \times 450 \times \pi \times 19^2 / 4}{1,25};\frac{0,29 \times 1 \times 19^2 \sqrt{25 \times 31000}}{1,25}\right) \times 10^{-3}$$

$$= 0,717 \times \mathrm{Min}(81,66\ \text{kN}\ ;\ 73,73\ \text{kN})$$

P_{Rd} = 52,86 kN

3.6.4. Degree of shear connection

The degree of shear connection is defined by: (EN 1994-1-1 § 6.2.1.3 (3))

$$\eta = \frac{N_c}{N_{c,f}}$$

where:

N_c is the design value of the compressive normal force in the concrete flange

$N_{c,f}$ is the design value of the compressive normal force in the concrete flange with full shear connection

At mid-span the compressive normal force in the concrete flange represents the total connection.

A_c is the cross-sectional area of concrete, so at mid-span $A_c = b_{eff}\, h_c$

h_c $= h - h_p = 120 - 58 = 62$ mm

$\therefore A_c$ $= 1875 \times 62 = 116250$ mm^2

$$N_{c,f} = 0,85 A_c f_{cd} = 0,85 A_c \frac{f_{ck}}{\gamma_c} = 0,85 \times 116250 \times \frac{25}{1,5} \times 10^{-3} = 1647\ \text{kN}$$

The resistance of the shear connectors limits the normal force to:

$N_c = 0,5\ n\ P_{Rd} = 0,5 \times 36 \times 52,86 = 952$ kN

$$\therefore\ \eta = \frac{N_c}{N_{c,f}} = \frac{952}{1647} = 0,578$$

The ratio η is less than 1,0 so the connection is partial.

3.6.5. Verification of bending resistance

Minimum degree of shear connection

The minimum degree of shear connection for a steel section with equal flanges is given by: EN 1994-1-1 § 6.6.1.2

$$\eta_{\min} = 1 - \left(\frac{355}{f_y}\right)(0{,}75 - 0{,}03 L_e) \text{ with } L_e \leq 25$$

L_e is the distance in sagging bending between points of zero bending moment in metres, in this example: $L_e = 7{,}5$ m

$\therefore\ \eta_{\min} = 1 - (355 / 355)\ (0{,}75 - 0{,}03 \times 7{,}50) = 0{,}475$

$\eta = 0{,}578 > \eta_{\min} = 0{,}475$ OK

Plastic Resistance Moment at mid span

The design value of the normal force in the structural steel section is: EN 1994-1-1 § 6.6.1.2 and § 6.2.1.3

$N_{pl,a} = A_a f_y / \gamma_{M0} = 4595 \times 355 \times 10^{-3} / 1{,}0 = 1631$ kN

$\therefore N_{pl,a} = 1631 \text{ kN} > N_c = 952$ kN

For ductile shear connectors and a Class 1 steel cross-section, the bending resistance, M_{Rd}, of the critical cross-section of the beam (at mid span) is calculated by means of rigid-plastic theory except that a reduced value of the compressive force in the concrete flange N_c is used instead of $N_{c,f}$.

The plastic stress distribution is shows in Figure A.4.

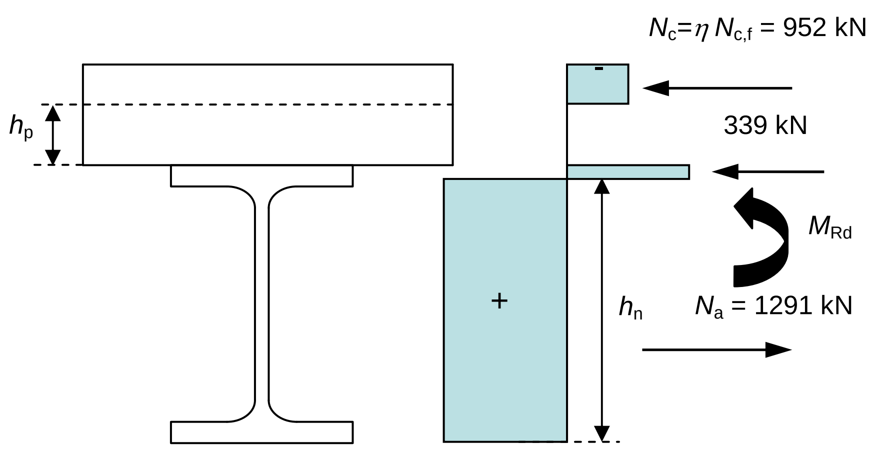


Figure A.4 Plastic stress distribution

The position of the neutral axis is: $h_n = 263$ mm

Therefore the design resistance for bending of the composite cross-section is:

$M_{Rd} = 301{,}7$ kNm

So, $M_{y,Ed} / M_{Rd} = 172{,}2 / 301{,}7 = 0{,}57 < 1$ OK

3.6.6. Shear Resistance

The shear plastic resistance depends on the shear area of the steel beam, which is given by:

EN 1993-1-1 § 6.2.6 (3)

$A_v = A - 2\,b\,t_f + (t_w + 2\,r)\,t_f$

$A_v = 4595 - 2 \times 135 \times 10{,}2 + (6{,}6 + 2 \times 15) \times 10{,}2 = 2214\ \text{mm}^2$

Shear plastic resistance

EN 1994-1-1 § 6.2.2.2

$$V_{\text{pl,Rd}} = \frac{A_v\,(f_y/\sqrt{3})}{\gamma_{M0}} = \frac{2214 \times (355/\sqrt{3})}{1{,}0}\,10^{-3} = 453{,}8\ \text{kN}$$

$V_{\text{Ed}} / V_{\text{pl,Rd}} = 91{,}80 / 453{,}8 = 0{,}202 < 1{,}0$ OK

Verification to shear buckling is not required when:

EN 1993-1-1 § 6.2.6 (6)

$h_w / t_w \leq 72\,\varepsilon / \eta$

η may be conservatively taken as 1,0

$h_w / t_w = (270 - 2 \times 10{,}2) / 6{,}6 = 37{,}8 < 72 \times 0{,}81 / 1{,}0 = 58{,}3$ OK

3.6.7. Longitudinal Shear Resistance of the Slab

The plastic longitudinal shear stresses is given by :

EN 1992-1-1 § 6.2.4 Figure 6.7

$$v_{\text{Ed}} = \frac{\Delta F_d}{h_f\,\Delta x}$$

where:

$\Delta x = 7{,}5 / 2 = 3{,}75$ m

The value of Δx is half the distance between the section where the moment is zero and the section where the moment is a maximum, so there are two areas for the longitudinal shear resistance of the slab.

$\Delta F_d = N_c / 2 = 951{,}56 / 2 = 475{,}8$ kN

$h_f = h - h_p = 120 - 58 = 62$ mm

$$v_{\text{Ed}} = \frac{\Delta F_d}{h_f\,\Delta x} = \frac{475{,}8 \times 10^3}{62 \times 3750} = 2{,}05\ \text{N/mm}^2$$

To prevent crushing of the compression struts in the concrete flange, the following condition should be satisfied:

$v_{\text{Ed}} < \nu f_{cd} \sin\theta_f \cos\theta_f$ with $\nu = 0{,}6[1 - f_{ck}/250]$ and $\theta_f = 45°$

$$v_{\text{Ed}} < 0{,}6 \times \left[1 - \frac{25}{250}\right] \times \frac{25}{1{,}5} \times 0{,}5 = 4{,}5\ \text{N/mm}^2 \quad \text{OK}$$

The following inequality should be satisfied for the transverse reinforcement:

$A_{sf} f_{yd} / s_f \geq v_{\text{Ed}}\,h_f / \cot\theta_f$ where $f_{yd} = 500 / 1{,}15 = 435\ \text{N/mm}^2$

Assume the spacing of the bars $s_f = 250$ mm and there is no contribution from the profiled steel sheeting:

$$A_{sf} \geq \frac{2{,}05 \times 62 \times 250}{435 \times 1{,}0} = 73{,}05 \text{ mm}^2$$

Take 10 mm diameter bars (78,5 mm^2) at 250 mm cross-centres extending over the effective concrete breadth.

3.7. Serviceability Limit State verification

EN 1990 § 6.5.3

3.7.1. SLS Combination

$G_k + Q_k = 9{,}80 + 7{,}50 = 17{,}30$ kN/m

Deflection due to $G_k + Q_k$: $w = \dfrac{5\,(G+Q)L^4}{384\,E\,I_y}$

I_y is calculated for the equivalent section, by calculating an effective equivalent steel area of the concrete effective area:

EN 1994-1-1 § 5.4.2.2

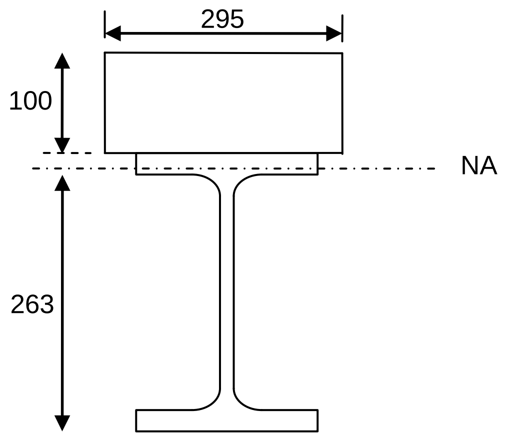


Figure A.5 Equivalent steel section used for the calculation of *A* and *I*$_y$

$b_{equ} = b_{eff} / n_0$

n_0 is the modular ratio for primary effects (Q_k)

$= E_a / E_{cm} = 210000 / 33000 = 6{,}36$

$\therefore\ b_{equ} = 1{,}875 / 6{,}36 = 0{,}295$ m

Using the parallel axis theorem the second moment of area is:

$I_y = 24\,540$ cm^4

For the permanent action:

n $= 2E_a / E_{cm} = 19{,}08$ for permanent loads (G_k)

EN 1994-1-1 § 5.4.2.2(11)

$\therefore\ b_{equ} = 1{,}875 / 19{,}06 = 0{,}0984$ m

The second moment of area is calculated as:

I_y $= 18\ 900\ \text{cm}^4$

The deflection can be obtained by combining the second moment of area for the variable and the permanent actions as follows:

$$w = \frac{5\ \times 7{,}5^4}{384\ \times 210000}\left(\frac{9{,}80}{18900 \times 10^{-8}} + \frac{7{,}50}{24540 \times 10^{-8}}\right) = \ 16 \text{ mm}$$

The deflection under ($G_k + Q_k$) is $L/469$

EN 1994-1-1 § 7.3.1

Note 1: The deflection limits should be specified by the client and the National Annex may specify some limits.

Note 2: The National Annex may specify limits concerning the frequency of vibration.

EN 1994-1-1 § 7.3.2

4. Simply Supported, Primary Composite Beam

This example shows the design of a composite beam of a multi-storey building, as shown in Figure A.6. The supporting beams are not propped and the profiled steel sheeting is parallel to the primary beam.

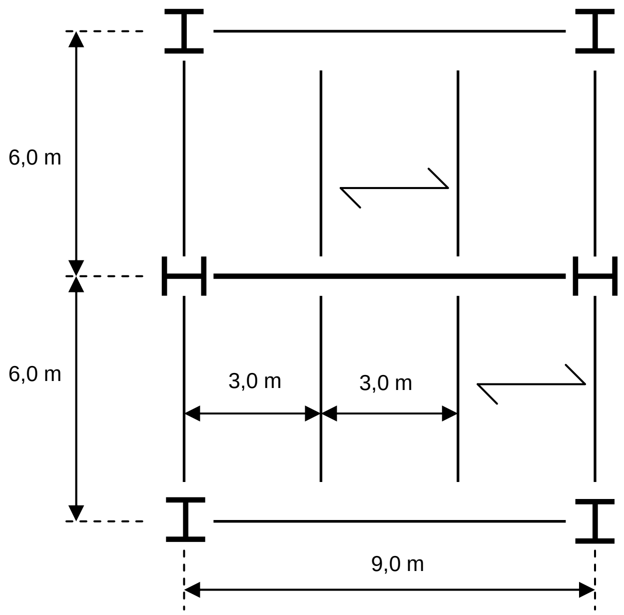
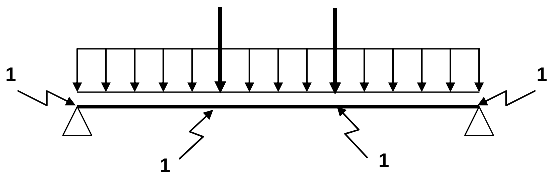


Figure A.6 Floor arrangement where the primary beam of this example is located

The secondary beams are represented by two concentrated loads as shown in Figure A.7:



1 Lateral restraints at the construction stage

Figure A.7 Loads applied to the primary beam

The beam is an I-rolled profile in bending about the major axis. This example includes:

- the classification of the cross-section
- the calculation of the effective width of the concrete flange
- the calculation of the shear resistance of a headed stud
- the calculation of the degree of shear connection
- the calculation of the bending resistance
- the calculation of the shear resistance
- the calculation of the longitudinal shear resistance of the slab
- the calculation of the deflection at serviceability limit state.

This example does not include any shear buckling verification of the web.

4.1. Partial factors

- γ_G = 1,35 (permanent loads)
- γ_Q = 1,50 (variable loads)
- γ_{M0} = 1,0
- γ_{M1} = 1,0
- γ_V = 1,25
- γ_C = 1,5

EN 1990

EN 1993-1-1, § 6.1 (1)

EN 1994-1-1, § 6.6.3.1

EN 1992-1-1

4.2. Basic data

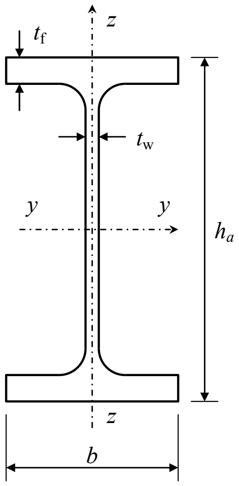
- Span length : 9,00 m
- Bay width : 6,00 m
- Slab depth : 14 cm
- Partitions : 0,75 kN/m^2
- Secondary beams (IPE 270) : 0,354 kN/m
- Imposed load : 2,50 kN/m^2
- Construction load : 0,75 kN/m^2
- Reinforced concrete density : 25 kN/m^3

4.3. Choose section

Try IPE 400 – Steel grade S355

4.3.1. Geometric data

Depth	h_a	= 400 mm
Width	b	= 180 mm
Web thickness	t_w	= 8,6 mm
Flange thickness	t_f	= 13,5 mm
Root radius	r	= 21 mm
Mass	66,3 kg/m	
Section area	A_a	= 84,46 cm^2



Second moment of area about the major axis	I_y	= 23130 cm^4
Elastic section modulus about the major axis	$W_{el,y}$	= 1156 cm^3
Plastic section modulus about the major axis	$W_{pl,y}$	= 1307 cm^3
Radius of gyration about the minor axis	i_z	= 3,95 cm
Modulus of elasticity of steel	E_a	= 210 000 N/mm^2

Yield strength

Steel grade S355

The maximum thickness is 13,5 mm < 40 mm, so: $f_y = 355$ N/mm^2

EN 1993-1-1, Table 3.1

Note: The National Annex may impose either the values of f_y from Table 3.1 or the values from the product standard.

Profiled steel sheeting

Thickness of sheet	t	= 0,75 mm
Slab depth	h	= 140 mm
Overall depth of the profiled steel sheeting excluding embossments	h_p	= 58 mm
Trapezoidal ribs	b_1	= 62 mm
	b_2	= 101 mm
	e	= 207 mm

Connectors

Diameter	d	= 19 mm
Overall nominal height	h_{sc}	= 100 mm
Ultimate tensile strength	f_u	= 450 N/mm^2
Number of studs	n	= 74 per row

(Stud at beam mid-span ignored)

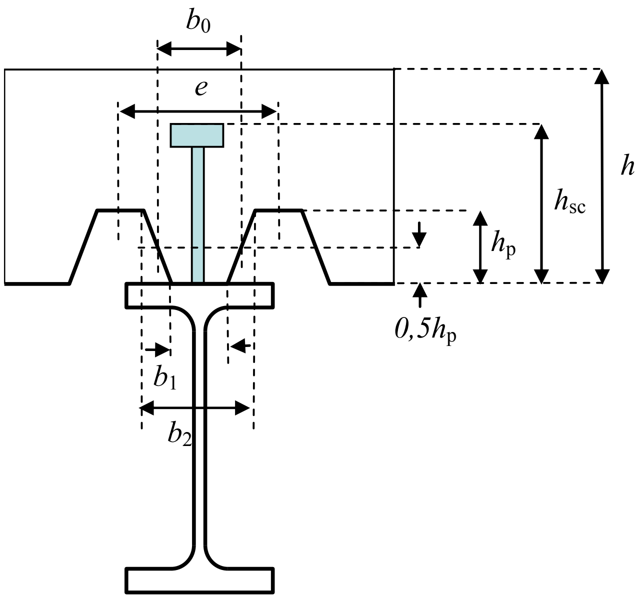


Figure A.8 Geometry of the composite beam

Concrete class: C 25/30

EN 1992-1-1, § 3.1.3 Table 3.1

Value of the compressive strength at 28 days $f_{ck} = 25$ N/mm^2

Secant modulus of elasticity of concrete $E_{cm} = 31\ 000$ N/mm^2

4.3.2. Actions on the beam at ULS

Permanent load:

To take into account the troughs of the profiled steel sheeting , the weight of the slab for the secondary beams is taken as:

$$25 \times 3{,}0 \times \left(0{,}14 - \frac{0{,}106 + 0{,}145}{2} \times \frac{0{,}058}{0{,}207}\right) = 7{,}86 \text{ kN/m}$$

Concentrated loads during the construction stage:

$F_G = (0{,}354 + 7{,}86) \times 6{,}0 = 49{,}28$ kN

Permanent loads in the final stage:

$F_G = (0{,}354 + 7{,}86 + 0{,}75 \times 3{,}0) \times 6{,}0 = 62{,}78$ kN

Self weight of the primary beam:

$q_G = 9{,}81 \times 66{,}3 \times 10^{-3} = 0{,}65$ kN/m

Variable load (Imposed load):

Concentrated loads during the construction stage:

$F_Q = 0{,}75 \times 3{,}0 \times 6{,}0 = 13{,}5$ kN

Concentrated loads in the final stage:

$F_Q = 2{,}5 \times 3{,}0 \times 6{,}0 = 45{,}0$ kN

ULS Combination:

EN 1990
§ 6.4.3.2

$\gamma_G F_G + \gamma_Q F_Q = 1,35 \times 62,78 + 1,50 \times 45,0 = 152,25$ kN

$\gamma_G q_G + \gamma_Q q_Q = 1,35 \times 0,65 = 0,877$ kN/m

Eq. (6.10) is used. In some countries, the National Annex may specify the use of equations (6.10a) and (6.10b).

ULS Combination during the construction stage:

$\gamma_G F_G + \gamma_Q F_Q = 1,35 \times 49,28 + 1,50 \times 13,5 = 86,78$ kN

$\gamma_G q_G + \gamma_Q q_Q = 1,35 \times 0,65 = 0,877$ kN/m

Bending moment diagram

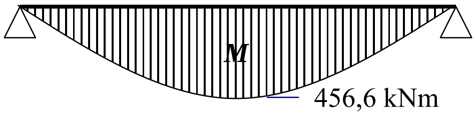


Figure A.9 Bending moment diagram at ULS in the final stage

Maximum moment at mid span:

$M_{y,Ed} = 3,0 \times 152,25 + 0,125 \times 0,877 \times 9,0^2 = 465,6$ kNm

Maximum moment at mid span (sequence of construction):

$M_{y,Ed} = 3,0 \times 86,78 + 0,125 \times 0,877 \times 9,0^2 = 269,2$ kNm

Shear force diagram

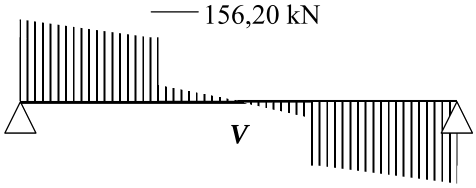


Figure A.10 Shear force diagram at ULS in the final stage

Maximum shear force at supports:

$V_{Ed} = 152,25 + 0,5 \times 0,877 \times 9,0 = 156,20$ kN

Maximum shear force at supports (sequence of construction):

$V_{Ed} = 86,78 + 0,5 \times 0,877 \times 9,0 = 90,73$ kN

4.3.3. Section classification:

The parameter ε is derived from the yield strength: $\varepsilon = \sqrt{\dfrac{235}{f_y}} = \sqrt{\dfrac{235}{355}} = 0,81$

Note: The classification is made for the non composite beam. For the composite beam the classification is more favourable for the web.

Outstand flange : flange under uniform compression

EN 1993-1-1 Table 5.2 (sheet 2 of 3)

$c \quad = (b - t_w - 2\,r) / 2 = (180 - 8{,}6 - 2 \times 21)/2 = 64{,}7$ mm

$c/t_f \quad = 64{,}7 / 13{,}5 = 4{,}79 \leq 9\,\varepsilon = 7{,}29$ Flange Class 1

Internal compression part

EN 1993-1-1 Table 5.2 (sheet 1 of 3)

$c \quad = h_a - 2\,t_f - 2\,r = 400 - 2 \times 13{,}5 - 2 \times 21 = 331$ mm

$c / t_w = 331 / 8{,}6 = 38{,}5 < 72\,\varepsilon = 58{,}3$ Web Class 1

The class of the cross-section is the highest class (i.e. the least favourable) of the flange and the web.

In this case the overall section is Class 1.

For Class 1 sections, the ULS verifications should be based on the plastic resistance of the cross-section.

4.3.4. Construction stage

Cross-sectional moment resistance

The design bending resistance of a cross-section is given by:

EN 1993-1-1 § 6.2.5

$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} f_y / \gamma_{M0} = (1307 \times 355 / 1{,}0) / 1000$

$M_{c,Rd} = 463{,}98$ kNm

$M_{y,Ed} / M_{c,Rd} = 269{,}2 / 463{,}98 = 0{,}58 < 1$ OK

Reduction factor for lateral–torsional buckling

To determine the design buckling resistance of a laterally unrestrained beam, the reduction factor for lateral–torsional buckling must be determined. The restraint provided by the steel sheet is in this case quite small and it is neglected. The following calculation determines this factor by a simplified method for lateral–torsional buckling. This method avoids calculating the elastic critical moment.

Non-dimensional slenderness

The non-dimensional slenderness may be obtained from the simplified method for steel grade S355:

SN002[4]

$$\bar{\lambda}_{LT} = \frac{L/i_z}{89} = \frac{300/3{,}95}{89} = 0{,}853$$

For rolled profiles, $\bar{\lambda}_{LT,0} = 0{,}4$

EN 1993-1-1, § 6.3.2.3(1)

Note: The value of $\bar{\lambda}_{LT,0}$ may be given in the National Annex. The recommended value is 0,4.

So $\bar{\lambda}_{LT} = 0{,}853 > \bar{\lambda}_{LT,0} = 0{,}4$

Reduction factor

For rolled sections, the reduction factor for lateral–torsional buckling is calculated from:

EN 1993-1-1 § 6.3.2.3 (1)

$$\chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta\,\bar{\lambda}_{LT}^2}} \quad \text{but} \quad \begin{cases} \chi_{LT} \le 1.0 \\ \chi_{LT} \le \dfrac{1}{\bar{\lambda}_{LT}^2} \end{cases}$$

where : $\phi_{LT} = 0{,}5\left[1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}\right) + \beta\,\bar{\lambda}_{LT}^2\right]$

α_{LT} is the imperfection factor for LTB. When applying the method for rolled profiles, the LTB curve has to be selected from Table 6.5:

EN 1993-1-1 Table 6.5 Table 6.3

For h_a/b = 400 / 180 = 2,22 > 2 Curve 'c' (α_{LT} = 0,49)

$\bar{\lambda}_{LT,0} = 0{,}4$ and $\beta = 0{,}75$

Note: The values of $\bar{\lambda}_{LT,0}$ and β may be given in the National Annex. The recommended values are 0,4 and 0,75 respectively.

We obtain : $\phi_{LT} = 0{,}5\left[1 + 0{,}49\left(0{,}853 - 0{,}4\right) + 0{,}75 \times (0{,}853)^2\right] = 0{,}884$

and : $$\chi_{LT} = \frac{1}{0{,}884 + \sqrt{(0{,}884)^2 - 0{,}75 \times (0{,}853)^2}} = 0{,}730$$

Then, we verify : χ_{LT} = 0,730 < 1,0

but : χ_{LT} = 0,730 < $1/\bar{\lambda}_{LT}^2$ = 1,374

So : χ_{LT} = 0,730

Design buckling resistance moment

$M_{b,Rd} = \chi_{LT}\, W_{pl,y} f_y / \gamma_{M1}$

EN 1993-1-1 § 6.3.2.1

$M_{b,Rd}$ = (0,730 × 1307000 × 355 / 1,0) × 10^{-6} = 338,7 kNm

$M_{y,Ed} / M_{b,Rd}$ = 269,2 / 338,7 = 0,795 < 1,0 OK

Shear Resistance

The shear plastic resistance depends on the shear area, which is given by:

EN 1993-1-1 § 6.2.6 (3)

$A_v = A - 2\,b\,t_f + (t_w + 2\,r)\,t_f$

A_v = 8446 – 2 × 180 × 13,5 + (8,6 + 2 × 21) × 13,5 = 4269 mm^2

Shear plastic resistance

EN 1993-1-1 § 6.2.6 (2)

$$V_{pl,Rd} = \frac{A_v\,(f_y/\sqrt{3})}{\gamma_{M0}} = \frac{4269 \times (355/\sqrt{3}) \times 10^{-3}}{1{,}0} = 874{,}97 \text{ kN}$$

$V_{Ed}/V_{pl,Rd}$ = 90,73 / 874,97 = 0,104 < 1,0 OK

Note that the verification to shear buckling is not required when : EN 1993-1-1 § 6.2.6 (6)

$h_w / t_w \leq 72\ \varepsilon / \eta$

The relevant value of η is : $\eta = 1{,}2$ EN 1993-1-5 § 5.1 (2)

$h_w / t_w = (400 - 2 \times 13{,}5) / 8{,}6 = 43 \quad < \quad 72 \times 0{,}81 / 1{,}2 = 48{,}6$

Interaction between bending moment and shear force

If $V_{Ed} < V_{pl,Rd} / 2$ then the shear force may be neglected. EN 1993-1-1 § 6.2.8 (2)

So, $V_{Ed} = 90{,}73$ kN $< V_{pl,Rd} / 2 = 874{,}97 / 2 = 437{,}50$ kN OK

4.3.5. Final stage

Effective width of concrete flange

The effective width is constant between 0,25 L and 0,75 L, where L is the span length. From L/4 to the closest support, the effective width decreases linearly. The concentrated loads are located between 0,25 L and 0,75 L. EN 1994-1-1 § 5.4.1.2

The total effective width is determined by: (Figure 5.1)

$$b_{\mathrm{eff},1} = b_0 + \sum b_{\mathrm{ei}}$$

b_0 is the distance between the centres of the outstand shear connectors, in this case $b_0 = 0$

b_{ei} is the value of the effective width of the concrete flange on each side of the web and taken as $b_{\mathrm{ei}} = L_e / 8$ but $\leq b_i = 3{,}0$ m

$b_{\mathrm{eff},1} = 0 + 9{,}0 / 8 = 1{,}125$ m, then $b_{\mathrm{eff}} = 2 \times 1{,}125 = 2{,}25$ m $< 3{,}0$ m

Design shear resistance of a headed stud

The shear resistance should be determined by: EN 1994-1-1 § 6.6.3.1

$$P_{\mathrm{Rd}} = k_l \times \mathrm{Min}\left(\frac{0{,}8 f_u \pi\, d^2 / 4}{\gamma_V} ; \frac{0{,}29 \alpha\, d^2 \sqrt{f_{ck}\, E_{cm}}}{\gamma_V} \right)$$

$h_{sc} / d = 100 / 19 = 5{,}26 > 4$, so $\alpha = 1$

Reduction factor (k_l)

For sheeting with ribs transverse to the supporting beam, the reduction factor for shear resistance is calculated by: EN 1994-1-1 § 6.6.4.1

$$k_l = 0{,}6 \frac{b_0}{h_p} \left(\frac{h_{sc}}{h_p} - 1 \right) \text{ but } \leq 1$$

where:

n_r = 1

h_p = 58 mm

h_{sc} = 100 mm

b_0 = 82 mm

$$\therefore \; k_1 = 0{,}6\frac{82}{58}\left(\frac{100}{58}-1\right) = 0{,}614 \leq 1{,}0 \quad \text{OK}$$

$$P_{\text{Rd}} = 0{,}614 \times \text{Min}\left(\frac{0{,}8\times 450\times \pi \times 19^2/4}{1{,}25}\times 10^{-3};\frac{0{,}29\times 1\times 19^2\sqrt{25\times 31000}}{1{,}25}\times 10^{-3}\right)$$

$$= 0{,}614 \times \text{Min}\left(81{,}66;\, 73{,}73\right) = 45{,}27 \text{ kN}$$

Degree of shear connection

The degree of shear connection is defined by:

EN 1994-1-1 § 6.2.1.3 (3)

$$\eta = \frac{N_c}{N_{c,f}}$$

where:

N_c is the design value of the compressive normal force in the concrete flange

$N_{c,f}$ is the design value of the compressive normal force in the concrete flange with full shear connection

At the load location:

The compressive normal force in the concrete flange represents the force for full connection.

A_c is the cross-sectional area of concrete, so at the load location:

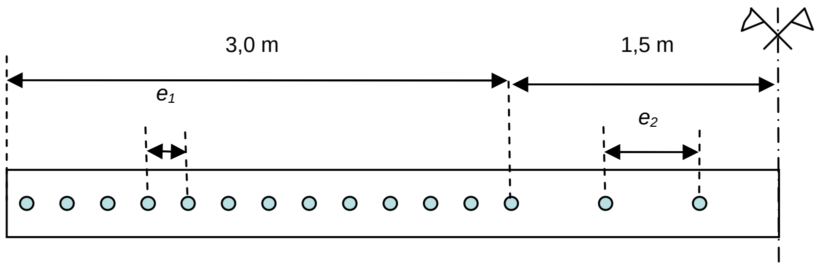
A_c = $b_{eff}\, h_c$

h_c = h - h_p = 140 – 58 = 82 mm

$\therefore A_c$ = 2250 × 82 = 184500 mm^2

$$\therefore \; N_{c,f} = 0{,}85 A_c f_{cd} = 0{,}85 A_c \frac{f_{ck}}{\gamma_C} = 0{,}85 \times 184500 \times \frac{25}{1{,}5} 10^{-3} = 2614 \text{ kN}$$

Since the maximum moment is reached nearly at the load location, the studs should be placed between the support and the concentrated load. However, studs should also be placed between the concentrated loads.



31 studs spaced at e_1 = 95 mm and 6 studs spaced at e_2 = 220 mm

Figure A.11 Location of studs

So, the resistance of the shear connectors limits the normal force to not more than:

$$N_c = n \times P_{Rd} = 31 \times 45,27 = 1403 \text{ KN}$$

$$\therefore \ \eta = \frac{N_c}{N_{c,f}} = \frac{1403}{2614} = 0,537$$

The ratio η is less than 1,0 so the connection is partial.

Verification of bending resistance

Minimum degree of shear connection

The minimum degree of shear connection for a steel section with equal flanges is given by

EN 1994-1-1 § 6.6.1.2

$$\eta_{min} = 1 - \left(\frac{355}{f_y}\right)(0,75 - 0,03L_e) \text{ with } L_e \leq 25 \text{ m}$$

L_e is the distance in sagging bending between points of zero bending moment in metres, for our example: $L_e = 9,0$ m

$\therefore \ \eta_{min} = 1 - (355 / 355)(0,75 - 0,03 \times 9,0) = 0,520$

$\therefore \ \eta_{min} = 0,520 < \eta = 0,537$ OK

Plastic resistance at the load location

The design value of the normal force in the structural steel section is:

EN 1994-1-1 § 6.2.1.2 and § 6.2.1.3

$$N_{pl,a} = A_a f_y / \gamma_{M0} = 8446 \times 355 \times 10^{-3} / 1,0 = 2998 \text{ kN}$$

$$\therefore \ N_{pl,a} > N_c = \eta \times N_{c,f} = 0,537 \times 2614 = 1403 \text{ kN}$$

For ductile shear connectors and Class 1 cross-section of the steel beam, the resistance of the cross-section of the beam, M_{Rd}, at the load location is calculated by means of rigid-plastic theory except that a reduced value of the compressive force in the concrete flange, N_c, is used instead of N_{cf}.

The plastic stress distribution is shown in Figure A.12:

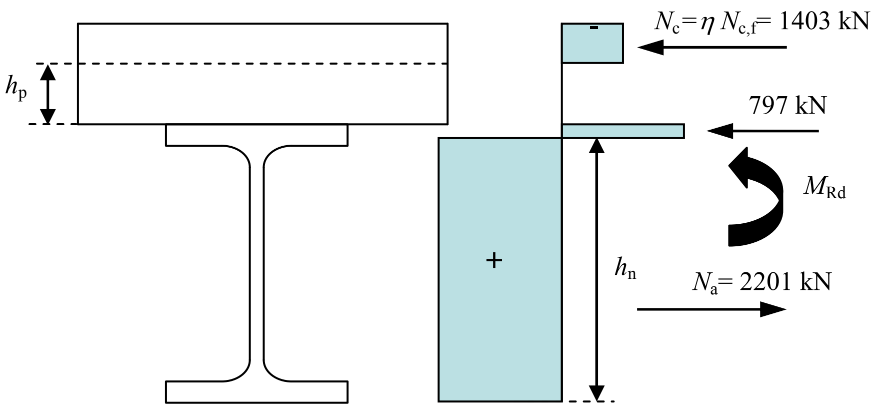


Figure A.12 Plastic stress distribution on the beam

The position of the plastic neutral axis is: $h_n = 388$ mm

Therefore, the design bending resistance of the composite cross-section is:

$M_{Rd} = 738$ kNm

$M_{y,Ed} / M_{Rd} = 465{,}6 / 738 = 0{,}63 < 1{,}0$ OK

Shear Resistance

EN 1994-1-1 § 6.2.2.2

The plastic shear resistance is the same as for steel beam alone.

$V_{pl,Rd} = 874{,}97$ kN

$V_{Ed} / V_{pl,Rd} = 156{,}20 / 874{,}97 = 0{,}18 < 1{,}0$ OK

Interaction between bending moment and shear force

EN 1993-1-1 § 6.2.8 (2)

If $V_{Ed} < V_{pl,Rd} / 2$ then the shear force may be neglected.

So, $V_{Ed} = 156{,}20$ kN $< V_{pl,Rd} / 2 = 874{,}97 / 2 = 437{,}50$ kN OK

Longitudinal Shear Resistance of the Slab

EN 1992-1-1 § 6.2.4 (Figure 6.7)

The plastic longitudinal shear stresses is given by :

$$v_{Ed} = \frac{\Delta F_d}{h_f \Delta x}$$

where $\Delta x = 9{,}0 / 3 = 3{,}0$ m

The value for Δx is the distance between the restraint and the point load. Therefore there are three areas for the longitudinal shear resistance.

$\Delta F_d = N_c / 2 = 1403 / 2 = 701{,}5$ kN

$h_f = h - h_p = 140 – 58 = 82$ mm

$$v_{Ed} = \frac{\Delta F_d}{h_f\, \Delta x} = \frac{701{,}5 \times 10^3}{82 \times 3000} = 2{,}85 \text{ N/mm}^2$$

To prevent crushing of the compression struts in the concrete flange, the following condition should be satisfied:

$v_{Ed} < \nu\, f_{cd} \sin\theta_f \cos\theta_f$ with $\nu = 0{,}6[1 - f_{ck}/250]$ and $\theta_f = 45°$

$$v_{Ed} < 0{,}6 \times \left[1 - \frac{25}{250}\right] \times \frac{25}{1{,}5} \times 0{,}5 = 4{,}5 \text{ N/mm}^2 \quad \text{OK}$$

The following inequality should be satisfied for the transverse reinforcement :

$A_{sf} f_{yd} / s_f \;\geq\; v_{Ed}\, h_f / cot\, \theta_f$ where $f_{yd} = 500 / 1{,}15 = 435$ N/mm^2

Assume the spacing of the bars $s_f = 200$ mm and there is no contribution from the profiled steel sheeting

$$A_{sf} \geq \frac{2{,}85 \times 82 \times 200}{435 \times 1{,}0} = 107{,}4 \text{ mm}^2$$

Take 12 mm diameter bars (113 mm^2) at 200 mm spacing.

4.4. Serviceability Limit State verifications

The deflection due to $G + Q$ is calculated as:

$$w_G = \frac{5\, q_G L^4}{384\, E\, I_y} + \frac{a \times (3L^2 - 4a^2)}{24\, E\, I_y} F_G$$

$$w_Q = \frac{a \times (3L^2 - 4a^2)}{24\, E\, I_y} F_Q$$

And the total deflection is: $w = w_G + w_Q$

4.4.1. Construction stage

EN 1990 § 6.5.3

SLS Combination during the construction stage

$F_G + F_Q = 49{,}28 + 13{,}5 = 62{,}78$ kN

$q_G = 0{,}65$ kN/m

Deflection during the construction stage

I_y is the second moment of area of the steel beam.

$$w_G = \frac{5 \times 0{,}65 \times 9000^4}{384 \times 210000 \times 23130 \times 10^4} + \frac{3000 \times (3 \times 9000^2 - 4 \times 3000^2)}{24 \times 210000 \times 23130 \times 10^4} \times 49280$$

$$w_G = 1{,}1 + 26{,}2 = 27{,}3 \text{ mm}$$

$$w_Q = \frac{3000 \times (3 \times 9000^2 - 4 \times 3000^2)}{24 \times 210000 \times 23130 \times 10^4} \times 13500 = 7{,}2 \text{ mm}$$

$\therefore\ w = w_G + w_Q = 27{,}3 + 7{,}2 = 34{,}5$ mm

The deflection under ($G + Q$) is L/261

Deflection in the final stage

$F_G + F_Q = 62,78 + 45,0 = 107,78$ kN — EN 1990 § 6.5.3

$q_G = 0,65$ kN/m

Deflection at the final stage:

I_y is calculated for the equivalent section, by calculating an effective equivalent steel area of the concrete effective area.

$b_{equ} = b_{eff} / n_0$

n_0 is the modular ratio for primary effects (Q_k) — EN 1994-1-1 § 5.4.2.2

$= E_a / E_{cm} = 210000 / 31000 = 6,77$

$\therefore\ b_{eq} = 2,25 / 6,77 = 0,332$ m

Using the parallel axis theorem the second moment of area is obtained:

$I_y = 82458$ cm^4

For the permanent action:

$n = 2E_a / E_{cm} = 20,31$ for permanent loads (G_k) — EN 1994-1-1 § 5.4.2.2(11)

$\therefore\ b_{equ} = 2,25 / 20,31 = 0,111$ m

The second moment of area is calculated as:

$I_y = 62919$ cm^4

The deflection can be obtained by combining the second moment of area for the variable and the permanent actions as follows:

$w_G = 27,3$ mm

$$w_{partitions} = \frac{3000 \times (3 \times 9000^2 - 4 \times 3000^2)}{24 \times 210000 \times 62919 \times 10^4} \times 13500 = 2,6 \text{ mm}$$

$$w_Q = \frac{3000 \times (3 \times 9000^2 - 4 \times 3000^2)}{24 \times 210000 \times 82458 \times 10^4} \times 45000 = 6,7 \text{ mm}$$

So, $w = w_G + w_{partitions} + w_Q = 27,3 + 2,6 + 6,7 = 36,6$ mm

The deflection under ($G + Q$) is $L/246$ — EN 1994-1-1 § 7.3.1

Note 1: The deflection limits should be specified by the client and the National Annex may specify some limits.

Note 2: The National Annex may specify frequency limits. — EN 1993-1-1 § 7.2.3

5. Pinned Column Using Non-slender H Sections

This example shows how to carry out the design of a column of a multi-storey building. The column is an HE 300 B section in S235, and the restraints are positioned as shown in Figure A.13.

SN008

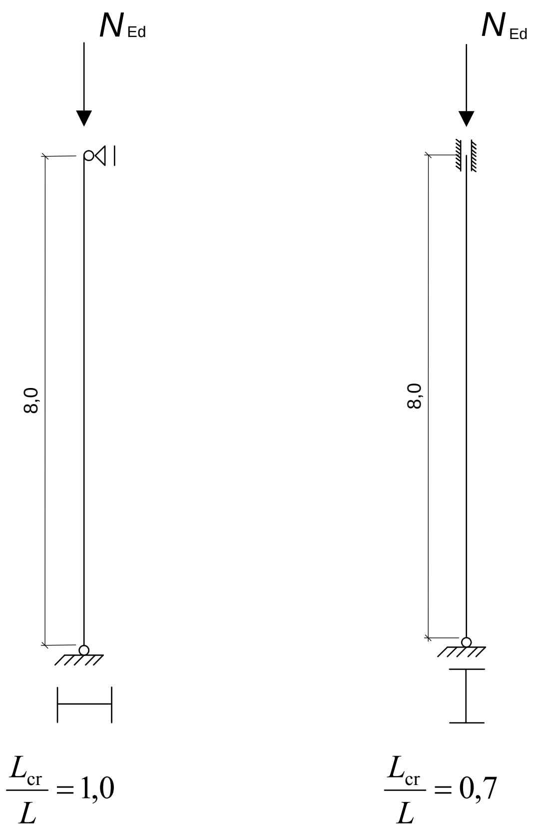


Figure A.13 End conditions of the column under consideration about the major and the minor axes and their buckling length factors.

5.1. Partial factors

EN 1993-1-1 § 6.1 (1)

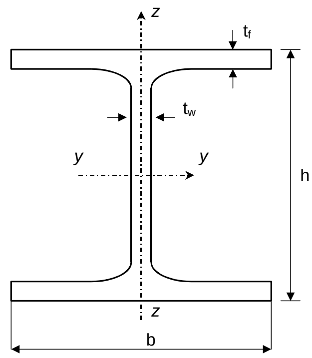
- $\gamma_{M0} = 1,0$
- $\gamma_{M1} = 1,0$

5.2. Basic data

- Axial load : $N_{Ed} = 2000$ kN
- Column length : 8,00 m
- Buckling length about the y-y axis: $1{,}0 \times 8{,}00 = 8{,}00\,\mathrm{m}$
- Buckling length about the z-z axis: $0{,}7 \times 8{,}00 = 5{,}60\,\mathrm{m}$
- Steel grade : S235
- Section classification: Class 1

5.3. Geometric properties of the section

HE 300 B – Steel grade S235



Depth h = 300 mm

Width b = 300 mm

Web thickness t_w = 11 mm

Flange thickness t_f = 19 mm

Root radius r = 27 mm

Section area A = 149 cm^2

Second moment of area about the major axis $I_y = 25170$ cm^4

Second moment of area about the minor axis $I_z = 8560$ cm^4

5.4. Yield strength

Steel grade S235

The maximum thickness is 19,0 mm < 40 mm, so: $f_y = 235$ N/mm^2

EN 1993-1-1
Table 3.1

5.5. Design buckling resistance of a compression member

To determine the design column buckling resistance $N_{b,Rd}$, the reduction factor χ for the relevant buckling curve must be obtained. This factor is determined by calculation of the non-dimensional slenderness $\bar{\lambda}$ based on the elastic critical force for the relevant buckling mode and the cross-sectional resistance to normal forces.

5.6. Elastic critical force N_{cr}

The critical buckling force is calculated as follows:

$$N_{cr,y} = \frac{\pi^2 \times EI_y}{L_{cr,y}^2} = \frac{\pi^2 \times 210000 \times 25170 \times 10^4}{8000^2} \times 10^{-3} = 8151{,}2\,\mathrm{kN}$$

$$N_{\mathrm{cr,z}} = \frac{\pi^2 \times EI_{\mathrm{z}}}{L_{\mathrm{cr,z}}{}^2} = \frac{\pi^2 \times 210000 \times 8560 \times 10^4}{5600^2} \times 10^{-3} = 5657{,}4 \text{ kN}$$

E is the modulus of elasticity = 210000 N/mm^2

L_{cr} is the buckling length in the buckling plane considered:

$L_{\mathrm{cr,y}}$ = 8,00 m

$L_{\mathrm{cr,z}}$ = 5,60 m

5.7. Non-dimensional slenderness

EN 1993-1-1 § 6.3.1.2 (1)

The non-dimensional slenderness is given by :

$$\bar{\lambda}_{\mathrm{y}} = \sqrt{\frac{Af_{\mathrm{y}}}{N_{\mathrm{cr,y}}}} = \sqrt{\frac{149 \times 10^2 \times 235}{8151{,}2 \times 10^3}} = 0{,}655$$

$$\bar{\lambda}_{\mathrm{z}} = \sqrt{\frac{Af_{\mathrm{y}}}{N_{\mathrm{cr,z}}}} = \sqrt{\frac{149 \times 10^2 \times 235}{5657{,}4 \times 10^3}} = 0{,}787$$

EN 1993-1-1 § 6.3.1.2 (4)

For slenderness $\bar{\lambda} \leq 0{,}2$ or for $\frac{N_{\mathrm{Ed}}}{N_{\mathrm{cr}}} \leq 0{,}04$ the buckling effects may be ignored and only cross-sectional verifications apply.

5.8. Reduction factor

For axial compression in members, the value of χ depending on the non-dimensional slenderness $\bar{\lambda}$ should be determined from the relevant buckling curve according to:

EN 1993-1-1 § 6.3.1.2 (1)

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1{,}0$$

where :

$$\phi = 0{,}5\left[1 + \alpha\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right]$$

α is the imperfection factor.

For h/b = 300/300 = 1,00 < 1,2 and t_{f} = 19,0 < 100 mm

Buckling about the y-y axis:

Buckling curve b, imperfection factor $\alpha = 0{,}34$

$$\phi_{\mathrm{y}} = 0{,}5\left[1 + 0{,}34\left(0{,}655 - 0{,}2\right) + 0{,}655^2\right] = 0{,}792$$

$$\chi_{\mathrm{y}} = \frac{1}{0{,}792 + \sqrt{0{,}792^2 - 0{,}655^2}} = 0{,}808$$

Buckling about the z-z axis:

Buckling curve *c*, imperfection factor $\alpha = 0{,}49$

$$\phi_z = 0{,}5\left[1 + 0{,}49\left(0{,}787 - 0{,}2\right) + 0{,}787^2\right] = 0{,}953$$

$$\chi_z = \frac{1}{0{,}953 + \sqrt{0{,}953^2 - 0{,}787^2}} = 0{,}671$$

$\chi = \min(\chi_y;\ \chi_z) = \min(0{,}808;\ 0{,}671) = 0{,}671 < 1{,}0$

(when $\chi > 1$ then $\chi = 1$)

5.9. Design buckling resistance of a compression member

EN 1993-1-1 § 6.3.1.1 (3)

$$N_{b,Rd} = \chi \frac{A \times f_y}{\gamma_{M1}} = 0{,}671 \frac{149 \times 10^2 \times 235}{1{,}0} \times 10^{-3} = 2349{,}5 \text{ kN}$$

EN 1993-1-1 § 6.3.1.1 (1)

The following expression must be verified:

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{2000}{2349{,}5} = 0{,}85 < 1{,}0 \qquad \text{OK}$$

6. Bolted connection of an angle brace in tension to a gusset

These types of connections are typical for cross bracings used both in façades and in roofs to withstand the actions of the horizontal wind load in the longitudinal axis of the single storey building. This is illustrated in SS048[4].

SS048[4]

In order to avoid eccentricities of the loads transferred to the foundation, the angle is aligned to meet the vertical axis of the column at the base plate. The gusset plate is placed as close as possible to the major axis plane of the column.

Table A.1 summarises the possible modes of failure in this connection. These verifications are shown in the following sections.

Table A.1 Modes of failure of the bracing connection

Mode of failure	Component Resistance
Bolts in shear	$N_{\mathrm{Rd},1}$
Bolts in bearing (on the angle leg)	$N_{\mathrm{Rd},2}$
Angle in tension	$N_{\mathrm{Rd},3}$
Weld design	a

6.1. Details of the bracing connection

Figure A.14 shows the long leg of the 120 × 80 angle that is attached to the gusset plate.

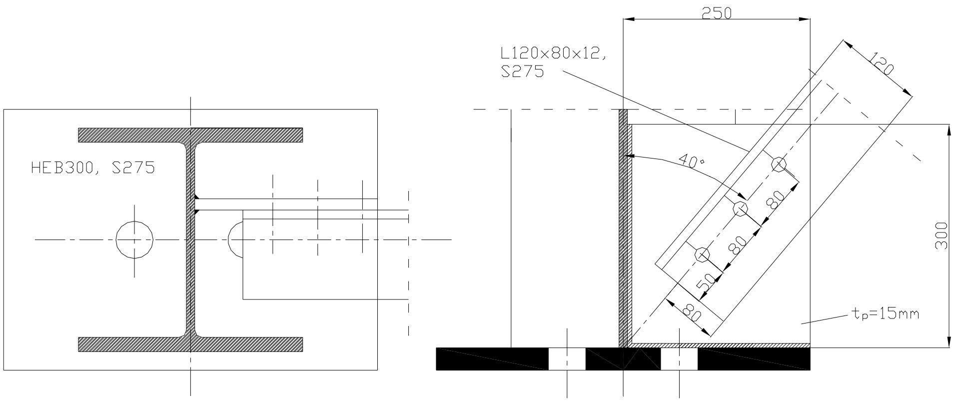


Figure A.14 Detail of the bolted connection: plan and elevation

Common practice is to minimize the eccentricity between the bracing member and the column axis. The gusset plate is welded to the column web and to the base plate using double fillet welds (see Figure A.14). Although there is some eccentricity in order to avoid the anchor bolt on the axis of the column, this is better than the bracing being on the plane of the column flange.

6.1.1. Main joint data

Configuration	Angle to gusset plate welded to a column web
Column	HEB 300, S275
Bracing	120 × 80 × 12 angle, S275
Type of connection	Bracing connection using angle to gusset plate and non-preloaded bolts Category A: Bearing type
Gusset plate	250 × 300 × 15, S275
Bolts	M20, class 8.8
Welds	Gusset plate to column web: fillet weld, $a = 4$ mm (see 6.2.4). Gusset plate to base plate: fillet weld, $a = 4$ mm (see 6.2.4).

6.1.2. Column HEB 300, S275

Depth	h_c	= 300 mm
Width	b_c	= 300 mm
Thickness of the web	$t_{w,c}$	= 11 mm
Thickness of the flange	$t_{f,c}$	= 19 mm
Fillet radius	r	= 27 mm
Area	A_c	= 149,1 cm^2
Second moment of area	I_y	= 25170 cm^4
Depth between fillets	d_c	= 208 mm
Yield strength	$f_{y,c}$	= 275 N/mm^2
Ultimate tensile strength	$f_{u,c}$	= 430 N/mm^2

6.1.3. Angle 120 × 80 × 12, S

Depth	h_{ac}	= 120 mm
Width	b_{ac}	= 80 mm
Thickness of the angle	t_{ac}	= 12 mm
Fillet radius	r_1	= 11 mm
Fillet radius	r_2	= 5,5 mm
Area	A_{ac}	= 22,7 cm^2
Second moment of area	I_y	= 322,8 cm^4
Yield strength	$f_{y,ac}$	= 275 N/mm^2
Ultimate tensile strength	$f_{u,ac}$	= 430 N/mm^2

6.1.4. Gusset plate 250 × 300 × 15, S275

Depth	h_p	= 300 mm
Width	b_p	= 250 mm
Thickness	t_p	= 15 mm
Yield strength	$f_{y,p}$	= 275 N/mm^2
Ultimate tensile strength	$f_{u,p}$	= 430 N/mm^2

Direction of load transfer (1)

Number of bolt rows	n_1	= 3
Angle edge to first bolt row	e_1	= 50 mm
Pitch between bolt rows	p_1	= 80 mm

Direction perpendicular to load transfer (2)

Number of lines of bolts	n_2	= 1
Angle attached leg edge to bolt line	e_2	= 80 mm

6.1.5. Bolts M20, 8.

Total number of bolts	$(n = n_1 \times n_2)$	$n = 3$
Tensile stress area	A_s	= 245 mm^2
Diameter of the shank	d	= 20 mm
Diameter of the holes	d_0	= 22 mm
Diameter of the washer	d_w	= 37 mm
Yield strength	f_{yb}	= 640 N/mm^2
Ultimate tensile strength	f_{ub}	= 800 N/mm^2

6.1.6. Partial safety factors

γ_{M0} = 1,0

γ_{M2} = 1,25 (for shear resistance of bolts)

6.1.7. Design axial tensile force applied by the angle brace to the gusset plate

N_{Ed} = 250 kN

6.2. Resistance of the bracing connection

6.2.1. Bolts in shear

EN 1993-1-8 Table 3.4.

$$N_{Rd,1} = nF_{v,Rd}$$

$$F_{v,Rd} = \alpha_v \frac{f_{ub} A}{\gamma_{M,2}} = 0,6 \times \frac{800 \times 245}{1,25} \times 10^{-3} = 94,08\ \mathrm{kN}$$

$$N_{Rd,1} = 3 \times 94,08 = 282\ \mathrm{kN}$$

6.2.2. Bolts in bearing (on the angle leg)

EN 1993-1-8 Table 3.4.

$$N_{Rd,2} = nF_{b,Rd}$$

$$F_{b,Rd} = \frac{k_1 \alpha_b f_{u,ac} d t_{ac}}{\gamma_{M2}}$$

All bolts:

$$k_1 = \min\left(2,8 \times \frac{e_2}{d_0} - 1,7; \quad 2,5\right)$$

$$2,8 \times \frac{e_2}{d_0} - 1,7 = 2,8 \times \frac{80}{22} - 1,7 = 8,48$$

$$\therefore\ k_1 = \min(8,48; \quad 2,5) = 2,5$$

End bolt:

$$\alpha_b = \min\left(\frac{e_1}{3d_0}; \quad \frac{f_{ub}}{f_{u,ac}}; \quad 1,0\right)$$

$$\frac{e_1}{3d_0} = \frac{50}{3 \times 22} = 0,76$$

$$\frac{f_{ub}}{f_{u,ac}} = \frac{800}{430} = 1,86$$

$$\therefore\ \alpha_b = \min(0,76; \quad 1,86; \quad 1,0) = 0,76$$

$$F_{b,Rd,end\ bolt} = \frac{2,5 \times 0,76 \times 430 \times 20 \times 12}{1,25} \times 10^{-3} = 156,9\ \mathrm{kN}$$

Inner bolts:

$$\alpha_{\rm b} = \min\left(\frac{p_1}{3d_0} - \frac{1}{4};\quad \frac{f_{\rm ub}}{f_{\rm u,ac}};\quad 1{,}0\right)$$

$$\frac{p_1}{3d_0} - \frac{1}{4} = \frac{80}{3\times 22} - \frac{1}{4} = 0{,}96$$

$$\frac{f_{\rm ub}}{f_{\rm u,ac}} = \frac{800}{430} = 1{,}86$$

$$\therefore\ \alpha_{\rm b} = \min(0{,}96;\quad 1{,}86;\quad 1{,}0) = 0{,}96$$

$$\therefore\ F_{\rm b,Rd,interior\ bolt} = \frac{2{,}5\times 0{,}96\times 430\times 20\times 12}{1{,}25}\times 10^{-3} = 198{,}1\ \rm kN$$

The bearing strength of the end bolt and of the inner bolt is greater than the bolt shear strength. The minimum value of the bearing strengths of all bolts in the connection is adopted for all bolts.

EN 1993-1-8 § 3.7(1)

$$\therefore\ N_{\rm Rd,2} = 3\times 156{,}9 = 471\ \rm kN$$

Note: The angle leg thickness, 12 mm, being less than that of the gusset plate, 15 mm, and assuming an end distance of 50 mm or greater for the gusset plate, only the attached angle leg requires a design verification for bearing.

6.2.3. Angle in tension

EN 1993-1-8 § 3.10.3

$$N_{\rm Rd,3} = \frac{\beta_3 A_{\rm net} f_{\rm u}}{\gamma_{\rm M2}}$$

$$2{,}5d_0 = 2{,}5\times 22 = 55\ \rm mm$$

$$5d_0 = 5\times 22 = 110\ \rm mm$$

$$2{,}5d_0 < p_1 < 5d_0$$

β_3 can be determined by linear interpolation:

$$\therefore\ \beta_3 = 0{,}59$$

$$A_{\rm net} = A - t_{\rm ac} d_0 = 2270 - 12\times 22 = 2006\ \rm mm^2$$

$$\therefore\ N_{\rm Rd,3} = \frac{0{,}59\times 2006\times 430}{1{,}25}\times 10^{-3} = 407\ \rm kN$$

6.2.4. Weld design

The weld is designed as follows:

The gusset plate is welded to the column web and to the base plate using double fillet welds.

The procedure to determine the throat thickness of the double fillet welds is the same for the gusset plate/column web connection and for the gusset plate/base plate connection.

The following calculations show the design of the weld between the gusset plate and the base plate.

It is possible to provide full strength double fillet welds following simplified recommendations, see SN017[4]. However, that approach is too conservative for this example.

SN017[4]

The recommended procedure is to choose a weld throat and to verify whether it provides sufficient resistance:

In this case, try $a = 4\text{ mm}$.

Design resistance for the double weld, according to the simplified method:

EN 1993-1-8 § 4.5.3.3

$$N_{\mathrm{Rd,w,hor}} = 2F_{\mathrm{w,Rd}}l$$

$$F_{\mathrm{w,Rd}} = f_{\mathrm{vw,d}}a$$

$$f_{\mathrm{vw,d}} = \frac{f_{\mathrm{u}}/\sqrt{3}}{\beta_{\mathrm{w}}\gamma_{\mathrm{M2}}} = \frac{430/\sqrt{3}}{0,85\times 1,25} = 233,66\text{ N/mm}^2$$

$$\therefore\ F_{\mathrm{w,Rd}} = 233,66\times 4 = 934,6\text{ N/mm}$$

$$\therefore\ N_{\mathrm{Rd,w,hor}} = 2\times 934,6\times 250\times 10^{-3} = 467\text{ kN}$$

It supports the horizontal component of the force acting in the bracing:

$$N_{\mathrm{Ed,hor}} = N_{\mathrm{Ed}}\sin 40 = 250\times\sin 40 = 161\text{ kN}$$

Therefore the horizontal weld is OK.

The same approach can be used to design the vertical weld (the gusset plate is welded to the column web).

6.3. Summary

The following table summarizes the resistance values for the critical modes of failure. The governing value for the joint (i.e. the minimum value) is shown in bold type.

Table A.2 Summary of the resistance values in the bolted bracing connection

Mode of failure	Component resistance	
Bolts in shear	$N_{\mathrm{Rd,1}}$	**282 kN**
Bolts in bearing on the angle leg	$N_{\mathrm{Rd,2}}$	471 kN
Angle in tension	$N_{\mathrm{Rd,3}}$	407 kN

Some modes of failure have not been verified in this example, such as the gusset plate in bearing and in tension. These verifications are not necessary because the thickness of the gusset plate is greater than that of the angle, and therefore the angle cleat would fail before the plate.