STEEL BUILDINGS IN EUROPE

Single-Storey Steel Buildings
Part 6: Detailed Design of Built-up Columns

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FOREWORD

This publication is part six of the design guide, *Single-Storey Steel Buildings*.

The 11 parts in the *Single-Storey Steel Buildings* guide are:

Part 1: Architect's guide
Part 2: Concept design
Part 3: Actions
Part 4: Detailed design of portal frames
Part 5: Detailed design of trusses
Part 6: Detailed design of built-up columns
Part 7: Fire engineering
Part 8: Building envelope
Part 9: Introduction to computer software
Part 10: Model construction specification
Part 11: Moment connections

Single-Storey Steel Buildings is one of two design guides. The second design guide is *Multi-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 guide covers the structural arrangements and the calculations for built-up columns fabricated from hot rolled sections.

The calculations refer to the European Standard EN 1993-1-1, with complementary information where necessary.

The design procedures of EN 1993-1-1 are presented to verify a built-up column with lacing or battening using simplified equations and formulas.

A worked example is given in Appendix A.

1 INTRODUCTION

Built-up columns are used in steel construction when the column buckling lengths are large and the compression forces are relatively low. This guide covers two types of built-up columns:

- Built-up columns with lacing
- Built-up columns with battens.

This document includes an overview of common details for such members. It describes the design method according to EN 1993-1-1[1] for the determination of the internal forces and the buckling resistance of each member (chords, diagonals, etc) of built-up columns made of hot rolled profiles.

It should be noted that due to the shear deformation, battened built-up columns are more flexible than solid columns with the same inertia; this must be taken into account in the design.

In order to derive the axial resistance of a steel built-up column, the following must be addressed:

- Analysis of the built-up column to determine the internal forces by taking into account an equivalent initial imperfection and the second order effects
- Verification of the chords and bracing members (diagonals and battens)
- Verification of the connections.

A fully worked example of a built-up column with an N-shape arrangement of lacings is given in Appendix A, which illustrates the design principles.

2 TYPES OF BUILT-UP MEMBERS AND THEIR APPLICATION

2.1 General

In general, built-up columns are used in industrial buildings, either as posts for cladding when their buckling length is very long, or as columns supporting a crane girder.

When used as a post for cladding with pinned ends, the column is designed to support the horizontal forces, mainly due to wind. Hence the bending moment in such a built-up column is predominant compared to the compression force.

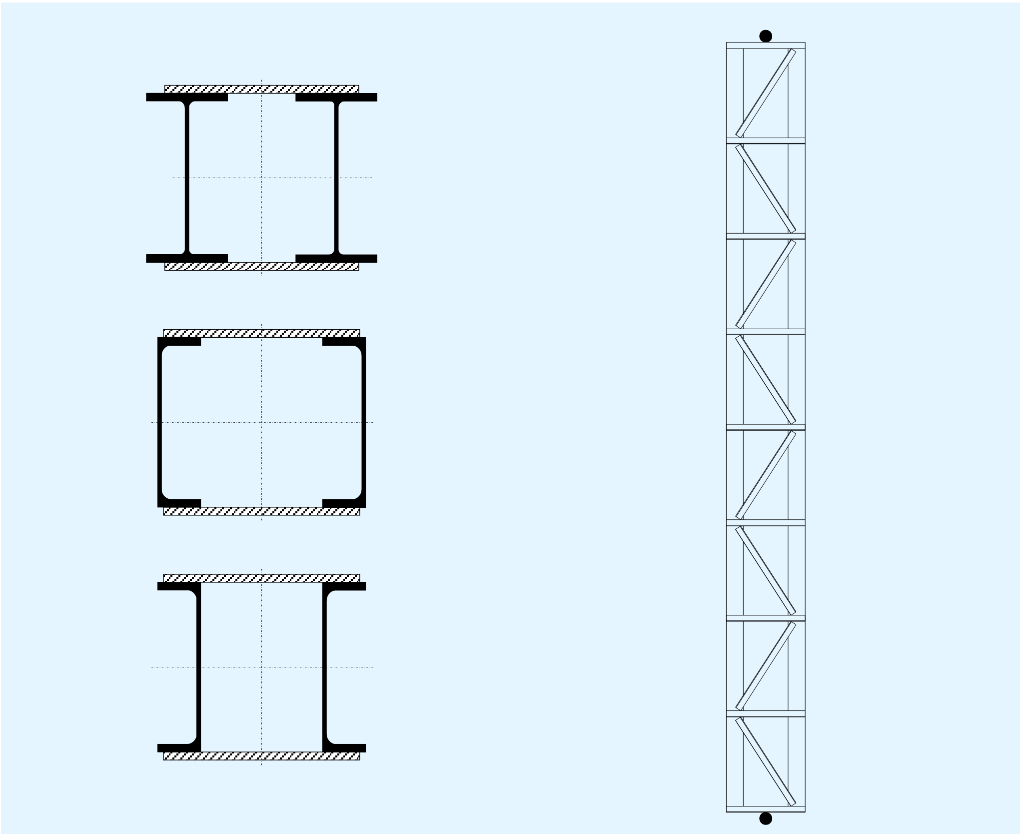
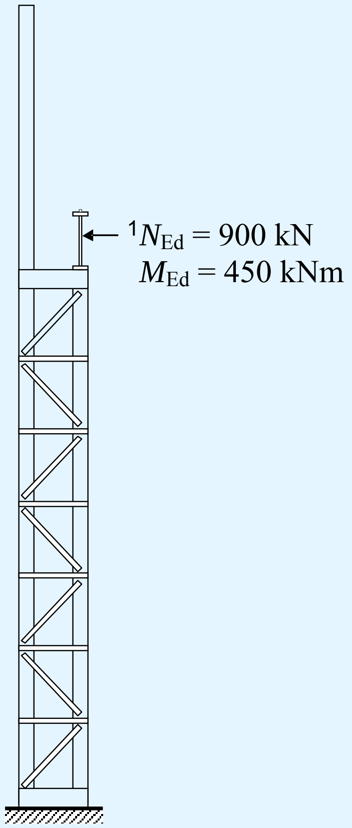


Figure 2.1 Post for cladding with pinned ends

A typical built-up column that supports a crane girder is shown in Figure 2.2. They usually have a fixed base and a pinned end at the top, and are designed to resist:

- The compression forces that result either from the frame or from the crane rail
- The horizontal forces that result from the effects of the crane applied on the internal chord and the wind loads applied to the external one.

In this case, the compression forces are predominant compared to the bending moment.



1 Crane girder

Figure 2.2 Built-up column supporting a crane girder

The built-up columns are composed of two parallel chords interconnected by lacings or battens – see Figure 2.1. In general, the truss system concentrates material at the structurally most efficient locations for force transfer.

In an industrial building and for a given height, built up columns theoretically have the least steel weight of any steel framing system.

Any hot rolled section can be used for the chords and the web members of built-up columns. However, channels or I-sections are most commonly used as chords. Their combination with angles presents a convenient technical solution for built-up columns with lacing or battens. Flat bars are also used in built-up column as battens.

This guide covers two types of built-up columns with pinned ends that are assumed to be laterally supported:

- Laced columns
- Battened columns.

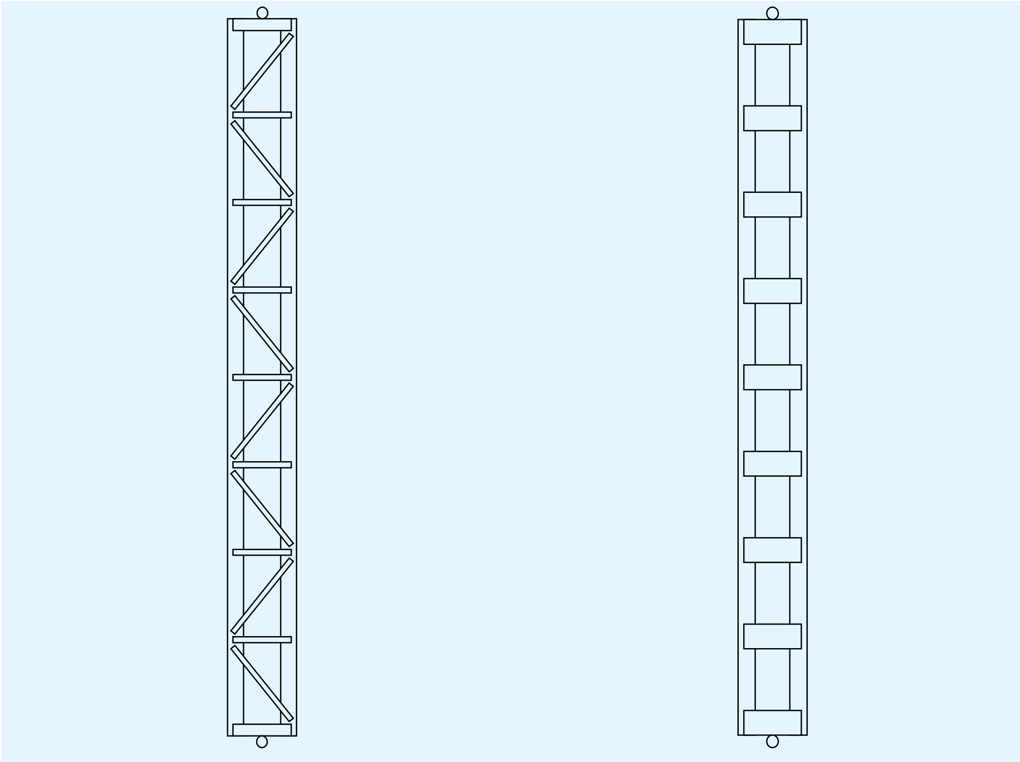


Figure 2.3 Built-up columns

The difference between these two types of built-up columns comes from the mode of connection of the web members (lacings and battens) to the chords. The first type contains diagonals (and possibly struts) designed with pinned ends. The second type involves battens with fixed ends to the chords and functioning as a rectangular panel.

The inertia of the built-up column increases with the distance between the chord axes. The increase in stiffness is counterbalanced by the weight and cost increase of the connection between members.

Built-up columns provide relatively light structures with a large inertia. Indeed, the position of the chords, far from the centroid of the built-up section, is very beneficial in producing a great inertia. These members are generally intended for tall structures for which the horizontal displacements are limited to low values (e.g. columns supporting crane girders).

The axial resistance of built-up columns is largely affected by the shear deformations. The initial bow imperfection is significantly amplified because of the shear strains.

It is possible to study the behaviour of built-up columns using a simple elastic model.

2.2 Laced built-up columns

2.2.1 General

There is a large number of laced column configurations that may be considered. However, the N-shape and the V-shape arrangements of lacings are commonly used.

Figure 2.4 Built-up column with lacings in an industrial building

The selection of either channels or I-sections for chord members provides different advantages. I-sections are more structurally efficient and therefore are potentially shallower than channels. For built-up columns with a large compressive axial force (for example, columns supporting cranes), I or H sections will be more appropriate than channels. Channels may be adequate in order to provide two flat sides.

Tee sections cut from European Column sections are also used for the chord members. The web of the Tee sections should be sufficiently deep to permit easy welding of the bracing members.

The angle web members of the laced column allow use of gusset-less welded connections, which minimises fabrication costs. Other member types require either gussets or more complex welding.

The centroidal axes of the compression and tension web members are not necessarily required to meet at the same point on the chord axes. In fact, laced columns with an eccentricity at the joints can be as efficient as those without eccentricity. The chord-web joint can be separated without an increase in steel weight. Although eccentric joints require that local moments be designed for, there are several advantages in doing so. Eccentric joints provide additional space for welding, hence reducing fabrication complexity. In addition, the reduced length of the compression chord provides enhanced buckling and bending resistance which partly compensates for the additional moments generated by the joint eccentricity. For single angles, it is recommended that joint eccentricity is minimised.

2.2.2 Various lacing geometries

The N-shape arrangement of lacings, as shown in Figure 2.5(a), can be considered as the most efficient truss configuration, for typical frames in industrial buildings. The web of the N-shape arrangement comprises diagonals and posts that meet at the same point on the chord axes.

This arrangement reduces the length of the compression chords and diagonals. It is usually used in frames with a significant uniform compressive force.

The V-shape arrangement of lacings increases the length of the compression chords and diagonals and provides a reduction of buckling resistance of the members. This arrangement is used in frames with a low compressive force.

The X-shape configurations are not generally used in buildings because of the cost and the complexity of fabrication.

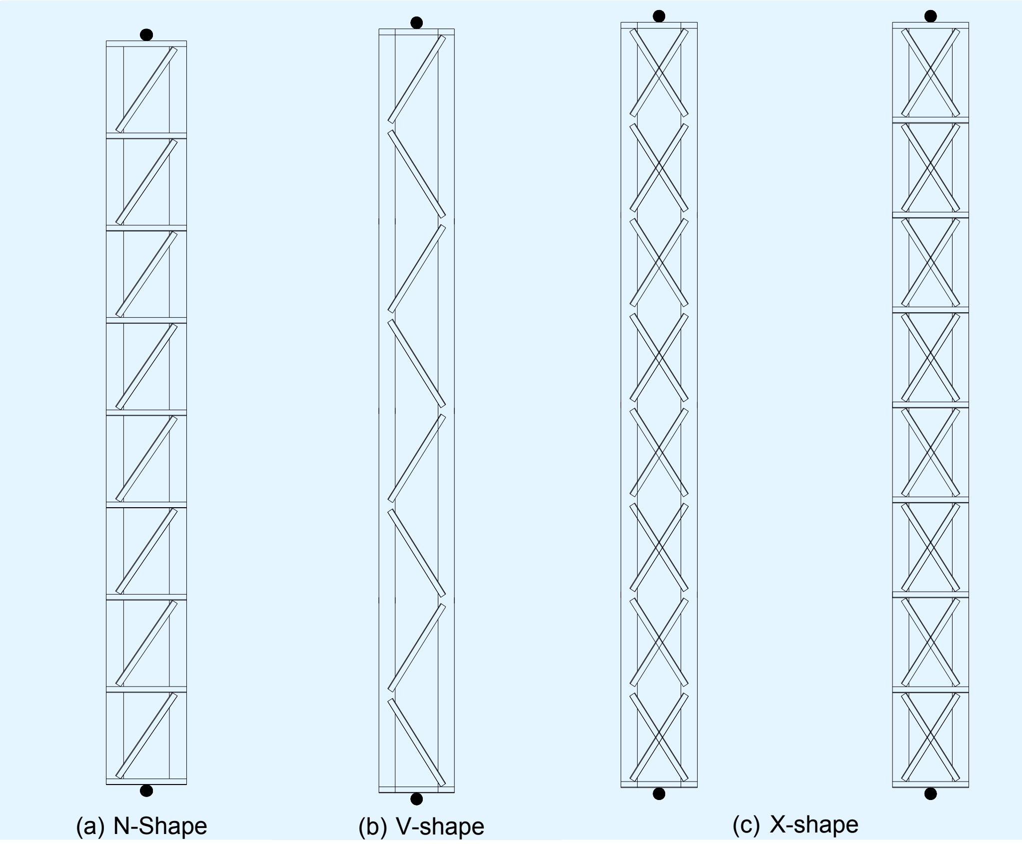


Figure 2.5 Different shape arrangements of lacing

2.2.3 Construction details

Single lacing systems on opposite faces of the built-up member with two parallel laced planes should be corresponding systems as shown in Figure 2.6(a) (EN 1993-1-1 § 6.4.2.2(1)).

When the single lacing systems on opposite faces of a built-up member with two parallel laced planes are mutually opposed in direction, as shown in Figure 2.6(b), the resulting torsional effects in the member should be taken into account. The chords must be designed for the additional eccentricity caused by the transverse bending effect, which can have a significant influence on the member size.

Tie panels should be provided at the ends of lacing systems, at points where the lacing is interrupted and at joints with other members.

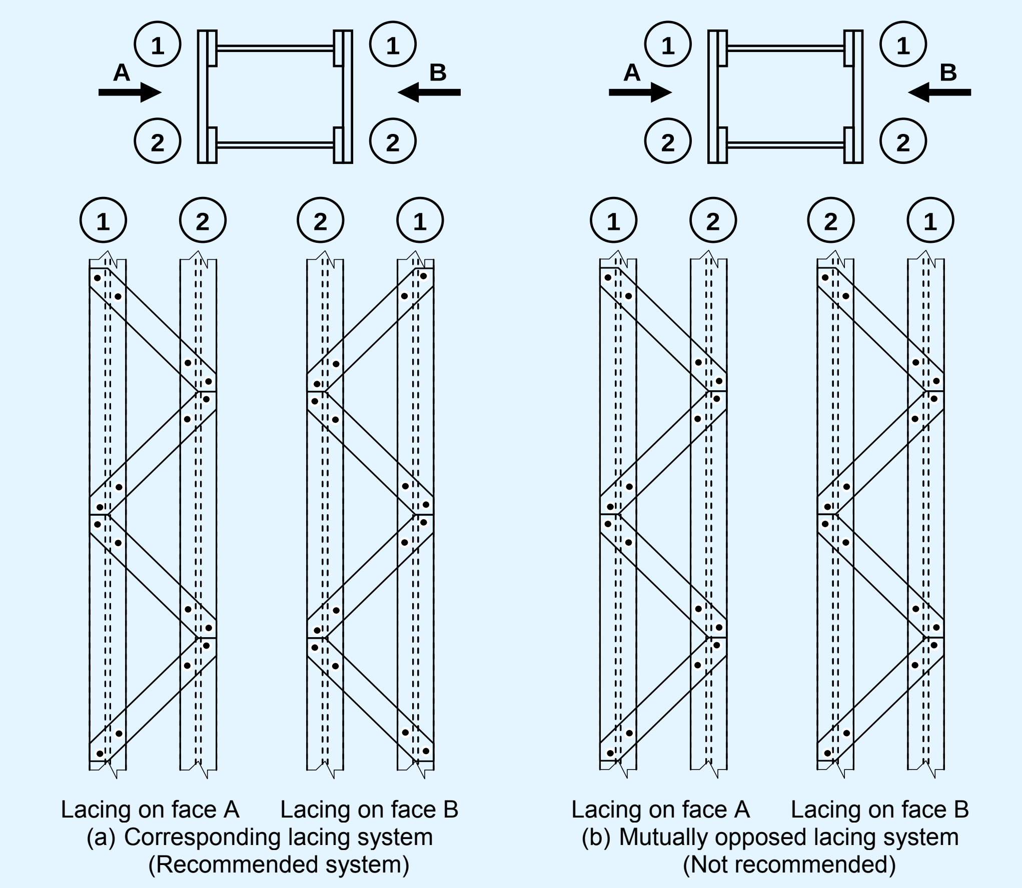


Figure 2.6 Single lacing system on opposite faces of a built-up member with two parallel laced planes

2.3 Battened built-up columns

Battened built-up columns are not appropriate for frames in industrial buildings. They are sometimes used as isolated frame members in specific conditions, where the horizontal forces are not significant.

Channels or I-sections are mostly used as chords and flat bars are used as battens. The battens must have fixed ends on the chords.

Battened built-up columns are composed of two parallel planes of battens which are connected to the flanges of the chords. The position of the battens should be the same for both planes. Battens should be provided at each end of the built-up member.

Battens should also be provided at intermediate points where loads are applied, and at points of lateral restraint.

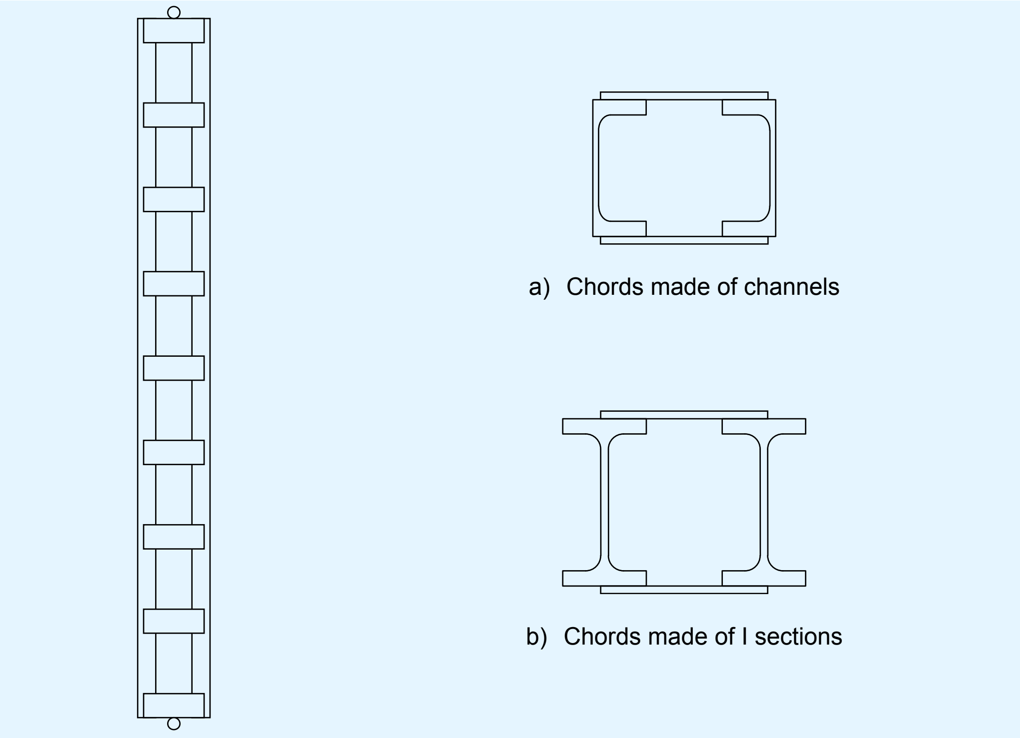


Figure 2.7 Battened compression members with two types of chords

3 DETAILED CALCULATIONS

3.1 General

The design methodology described hereafter can be applied to verify the resistance of the various components of a built-up member with pinned ends, for the most critical ULS combination. The design axial force, N_{Ed}, and the design bending moment, M_{Ed}, about the strong axis of the built-up member are assumed to have been determined from analysis in accordance with EN 1993-1-1[1].

This methodology is applicable to built-up columns where the lacing or battening consists of equal modules with parallel chords. The minimum number of modules in a member is three.

The methodology is summarized in the flowchart in Figure 3.2 for laced built-up columns, and in Figure 3.4 for battened built-up columns. It is illustrated by the worked example given in Appendix A.

3.2 Design methodology for laced built-up columns

3.2.1 Step 1: Maximum compression axial force in the chords

Effective second moment of area

The effective second moment of area is calculated using the following expression (EN 1993-1-1 § 6.4.2.1(4)):

$$I_{\mathrm{eff}} = 0{,}5\, h_0^2 A_{\mathrm{ch}}$$

where:

h_0 is the distance between the centroids of chords.

A_{ch} is the cross-sectional area of one chord.

Shear stiffness

For the stability verification of a laced built-up column, the elastic elongations of the diagonals and the posts must be considered in order to derive the shear stiffness S_v. Formulae for the shear stiffness S_v are given in Table 3.1 for different arrangements of lacing.

Initial bow imperfection

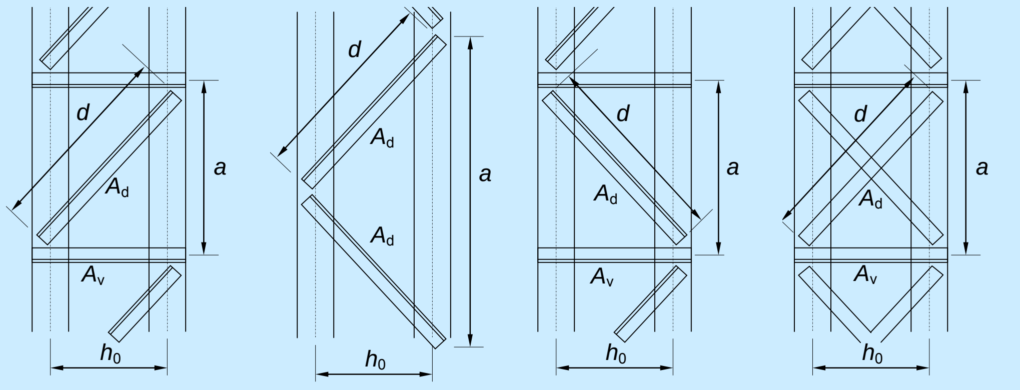
The built-up column is considered as a column with an initial bow imperfection of e_0, as shown in Figure 3.1:

$$e_0 = L/500$$

where:

L is the length of the built-up member

Table 3.1 Shear stiffness S_v of built-up columns



N-shape	V-shape	K-shape	X-shape
$S_V \dfrac{nEA_d a h_0^3}{d^3\left[1+\dfrac{A_d h_0^3}{A_d d^3}\right]}$	$S_V = \dfrac{nEA_d a h_0^2}{2d^3}$	$S_V = \dfrac{nEA_d a h_0^2}{d^3}$	$S_V = \dfrac{2nEA_d a h_0^2}{d^3}$

n is the number of planes of lacing
A_d is the section area of a diagonal
A_v is the section area of a post
d is the length of the diagonal

Figure 3.1 Initial bow imperfection

Maximum axial compression force in the chords

Verifications should be performed for chords using the design forces $N_{ch,Ed}$ resulting from the applied compression force N_{Ed} and the bending moment M_{Ed} at mid-height of the built-up column.

For a member with two identical chords, the design force $N_{ch,Ed}$ is determined from the following expression (EN 1993-1-1 § 6.4):

$$N_{ch,Ed} = \frac{N_{Ed}}{2} + \frac{M_{Ed} h_0 A_{ch}}{2I_{eff}}$$

where:

M_{Ed} is the maximum bending moment at mid-height of the built-up column including the equivalent imperfection e_0 and the second order effects:

$$M_{\mathrm{Ed}} = \frac{N_{\mathrm{Ed}} e_0 + M_{\mathrm{Ed}}^{\mathrm{I}}}{1 - \dfrac{N_{\mathrm{Ed}}}{N_{\mathrm{cr}}} - \dfrac{N_{\mathrm{Ed}}}{S_{\mathrm{v}}}}$$

N_{cr} is the effective critical force of the built-up column:

$$N_{\mathrm{cr}} = \frac{\pi^2 E I_{\mathrm{eff}}}{L^2}$$

N_{Ed} is the design compression axial force applied to the built-up column.

$M_{\mathrm{Ed}}^{\mathrm{I}}$ is the design value of the maximum moment at mid-height of the built-up column without second order effects.

3.2.2 Step 2: In-plane buckling resistance of the chord

Classification of the cross-section of the chord

The cross-section of the chord must be classified according to EN 1993-1-1 Table 5.2.

Buckling resistance of a chord about z-z axis

The resistance of the chord has to be verified for flexural buckling in the plane of the built-up member, i.e. about the weak axis of the cross-section of the chord (z-z axis). The buckling verification is given by (EN 1993-1-1 § 6.4.2):

$$\frac{N_{\mathrm{ch,Ed}}}{N_{\mathrm{b,z,Rd}}} \leq 1$$

where:

$N_{\mathrm{b,z,Rd}}$ is the design buckling resistance of the chord about the weak axis of the cross-section, calculated according to EN 1993-1-1 § 6.3.1.

Information on the buckling length L_{ch} of the chord is given in Section 3.4 of this guide.

3.2.3 Step 3: Out-of-plane buckling resistance of the chords

Out-of-plane buckling of the member, i.e. buckling about the strong axis of the cross-section of the chords (y-y axis), has to be considered. The buckling verification is given by:

$$\frac{N_{\mathrm{ch,Ed}}}{N_{\mathrm{b,y,Rd}}} \leq 1$$

where:

$N_{\mathrm{b,y,Rd}}$ is the design buckling resistance of the chord about the strong axis of the cross-section, calculated according to EN 1993-1-1 § 6.3.1.

The buckling length depends on the support conditions of the built-up member for out-of-plane buckling. At the ends of the member, the supports are generally considered as pinned. However intermediate lateral restraints may be provided.

3.2.4 Step 4: Maximum shear force

The verification of the web members of a built-up column with pinned ends is performed for the end panel by taking into account the shear force as described below.

For a built-up member subject to a compressive axial force only, the expression for the shear force is:

$$V_{\mathrm{Ed}} = \pi \frac{M_{\mathrm{Ed}}}{L}$$

where:

M_{Ed} is the bending moment as calculated in Step 2, with: $M_{\mathrm{Ed}}^{\mathrm{I}} = 0$

For a built-up member subject to a uniformly distributed load, the expression for the shear force is:

$$V_{\mathrm{Ed}} = 4 \frac{M_{\mathrm{Ed}}}{L}$$

where:

M_{Ed} is the maximum bending moment due to the distributed load.

Built-up columns are often subjected to a combination of a compressive axial force N_{Ed} and a uniformly distributed load. Thus the coefficient varies between π/L and $4/L$. As a simplification, the shear force may be calculated by linear interpolation:

$$V_{\mathrm{Ed}} = \frac{1}{L}\left(4 - (4-\pi)\frac{e_o N_{\mathrm{Ed}}}{e_o N_{\mathrm{Ed}} + M_{\mathrm{Ed}}^{I}}\right) M_{\mathrm{Ed}}$$

where:

M_{Ed} is the maximum bending moment as calculated in Step 2. The bending moment $M_{\mathrm{Ed}}^{\mathrm{I}}$ is the maximum moment due to the distributed load.

3.2.5 Step 5: Buckling resistance of the web members in compression

Maximum compressive axial force

The maximum axial force N_{Ed} in the web members adjacent to the ends is derived from the shear force V_{Ed}.

Classification of the web members in compression

The cross-section of the web member is classified according to EN 1993-1-1 Table 5.2.

Buckling resistance

The buckling verification of the web members should be performed for buckling about the weak axis of the cross-section, using the following criterion:

$$\frac{N_{\mathrm{ch,Ed}}}{N_{\mathrm{b,Rd}}} \leq 1$$

where, $N_{\mathrm{b,Rd}}$ is the design buckling resistance of the web member about the weak axis of the cross-section, calculated according to EN 1993-1-1 § 6.3.1.

Information about the buckling length of web members is given in Section 3.4.

3.2.6 Step 6: Resistance of the web members in tension

The resistance of the cross-section of the web members should be verified according to EN 1993-1-1 § 6.2.3 for the tensile axial force which is derived from the maximum shear force V_{Ed} as described in Step 3.

3.2.7 Step 7: Resistance of the diagonal-to-chord connections

The resistance of the connections between the web members and the chords has to be verified according to EN 1993-1-8[2]. This verification depends on the details of the connection: bolted connection or welded connection. This verification should be performed using the internal forces calculated in the previous steps.

The worked example in Appendix A includes the detailed verification of a welded connection.

3.2.8 Flowchart

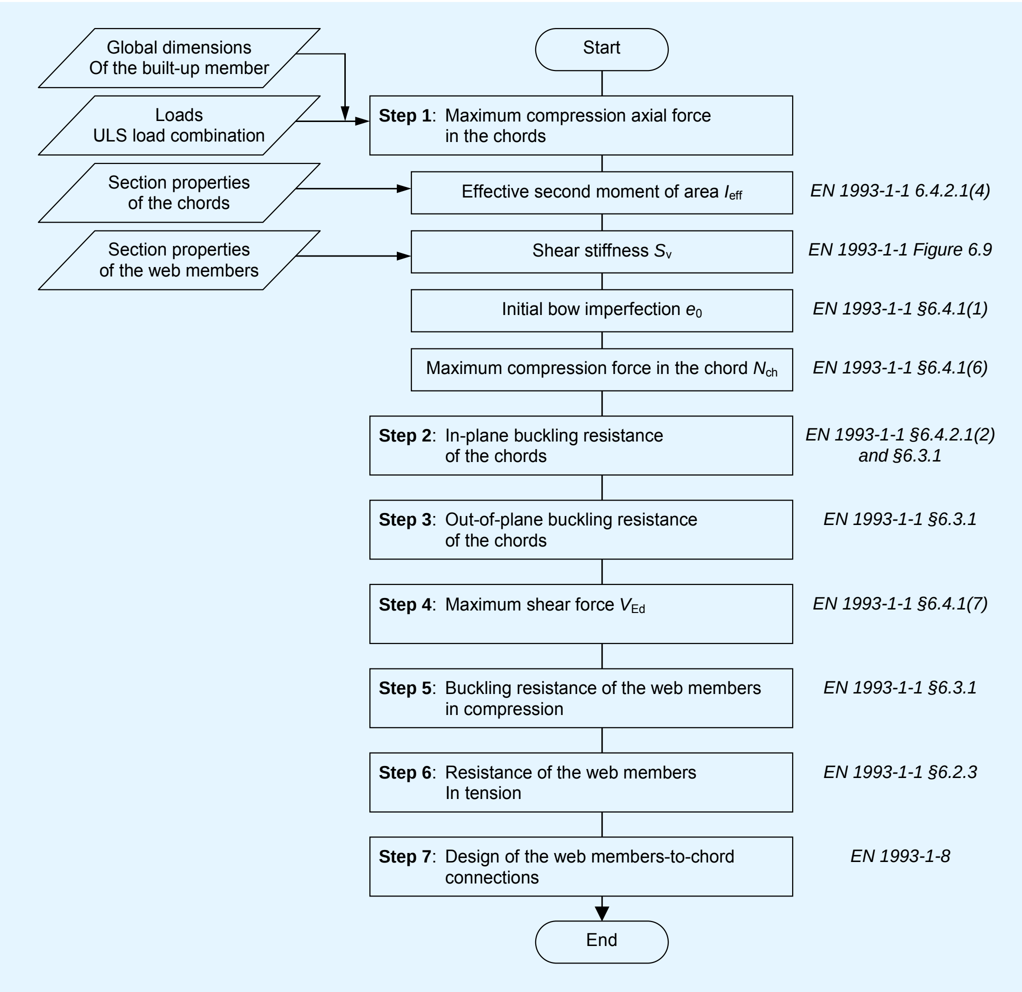


Figure 3.2 Flowchart of the design methodology for laced built-up columns

3.3 Design methodology for battened built-up columns

3.3.1 Step 1: Maximum compressive axial force in the chords

Effective second moment of area

The effective second moment of area is calculated using the following expression (EN 1993-1-1 § 6.4.3.1(3)):

$$I_{\text{eff}} = 0,5\, h_0^2 A_{\text{ch}} + 2\mu\, I_{\text{ch}}$$

where:

h_0 is the distance between the centroids of chords

A_{ch} is the cross-sectional area of one chord

I_{ch} is the in-plane second moment of area of one chord

μ is the efficiency factor according to Table 3.2.

Table 3.2 Efficiency factor (EN 1993-1-1 Table 6.8)

Criterion	Efficiency factor μ
$\lambda \geq 150$	0
$75 < \lambda < 150$	$2 - \lambda/75$
$\lambda \leq 75$	1,0
where: $\lambda = \dfrac{L}{i_0}$ $\quad i_0 = \sqrt{\dfrac{I_1}{2A_{\text{ch}}}}$	$I_1 = 0,5 h_0^2 A_{\text{ch}} + 2I_{\text{ch}}$

Shear stiffness

For the stability verification of a battened built-up column, the elastic deformations of the battens and the chords must be considered in order to derive the shear stiffness S_{v} using the following expression (EN 1993-1-1 § 6.4.3.1(2)):

$$S_v = \frac{24EI_{\text{ch}}}{a^2\left[1 + \dfrac{2I_{\text{ch}}}{nI_{\text{b}}}\dfrac{h_0}{a}\right]} \leq \frac{2\pi^2 EI_{\text{ch}}}{a^2}$$

But S_{v} should not be taken greater than $\dfrac{2\pi^2 EI_{\text{ch}}}{a^2}$

where:

a is the distance between the battens

n is the number of planes of battens

I_{b} is the in-plane second moment of area of one batten.

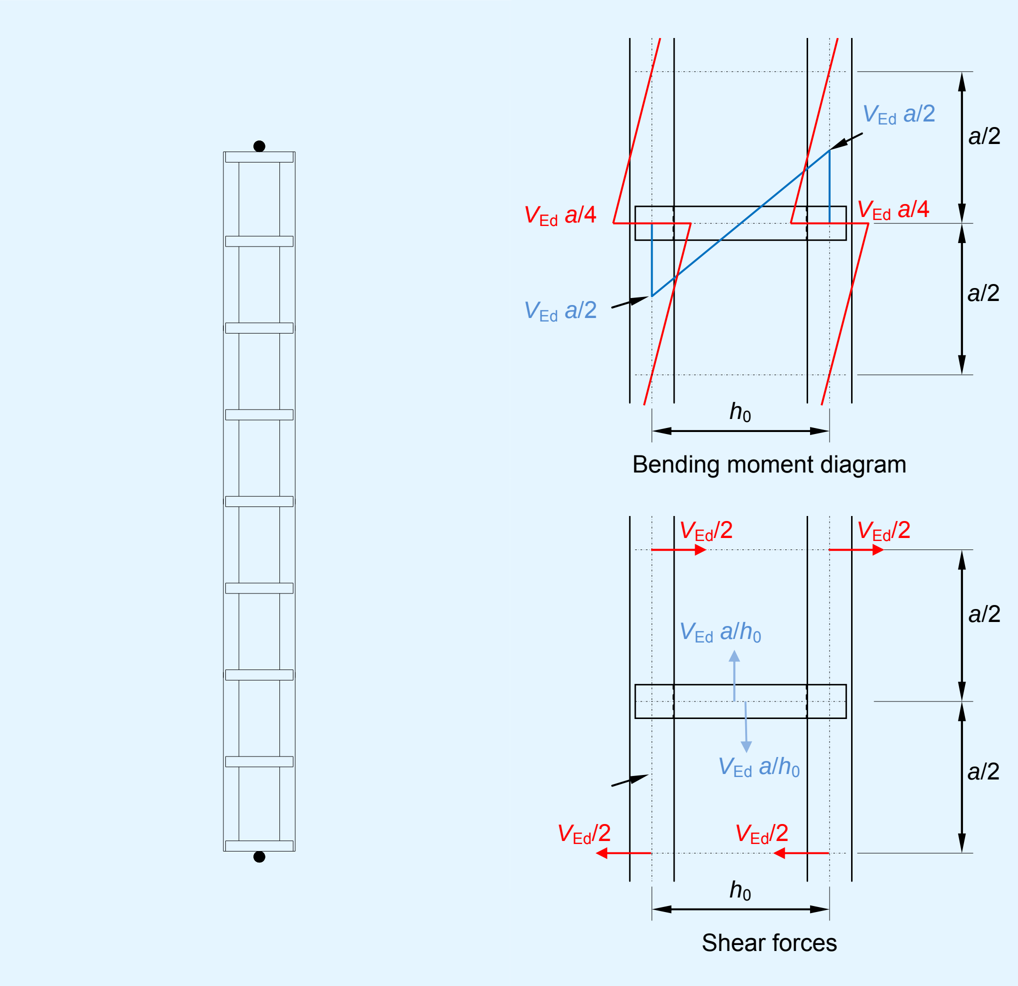


Figure 3.3 Bending moments and shear forces in a panel of a battened built-up column

Initial bow imperfection

The initial bow imperfection e_0 is:

$$e_0 = L/500$$

where:

L is the length of the built-up member

Maximum axial compressive force in the chords

The maximum axial compression $N_{ch,Ed}$ in the chords is calculated from the expression given in 3.2.1.

3.3.2 Step 2: In-plane buckling resistance of a chord

Classification of the cross-section of the chord

The cross-section of the chord is classified according to EN 1993-1-1 Table 5.2.

Buckling resistance of a chord about z-z axis

The resistance of the chord has to be verified for bending and axial compression and for buckling in the plane of the built-up member, i.e. about the weak axis of the cross-section of the chord (z-z axis), according to

EN 1993-1-1 § 6.3.3. Depending on the geometry of the battened built-up member, the verifications should be performed for different segments of the chord:

- For an end panel with the maximum shear force and thus the maximum local bending moment
- For a panel located at mid-height where the compression axial force may be maximum in the chord.

3.3.3 Step 3: Out-of-plane buckling resistance of the chords

The out-of-plane buckling resistance is verified using the following criterion:

$$\frac{N_{\mathrm{ch,Ed}}}{N_{\mathrm{b,y,Rd}}} \leq 1$$

where:

$N_{\mathrm{b,y,Rd}}$ is the design buckling resistance of the chord about the strong axis of the cross-section, calculated according to EN 1993-1-1 § 6.3.1.

The buckling length depends on the support conditions of the built-up member for out-of-plane buckling. At the ends of the member, the supports are generally considered as pinned. However intermediate lateral restraints may be provided.

3.3.4 Step 4: Shear force

The shear force V_{Ed} is calculated from the maximum bending moment as for a laced built-up member, according to §3.2.4 of this guide.

3.3.5 Step 5: Resistance of the battens

As shown in Figure 3.3, the battens should be designed to resist the shear force:

$$V_{\mathrm{Ed}} \frac{a}{h_0}$$

And the bending moment:

$$M_{\mathrm{Ed}} = \frac{V_{\mathrm{Ed}} a}{2}$$

The cross-section classification should be determined according to EN 1993-1-1 Table 5.2, for pure bending. The section resistance should be verified using the appropriate criteria given EN 1993-1-1 § 6.2.

3.3.6 Step 5: Resistance of the batten-to-chord connections

The resistance of the connections between the battens and the chords has to be verified according to EN 1993-1-8. This verification depends on the details of the connection: bolted connection or welded connection. This verification is performed using the internal forces calculated in the previous steps.

3.3.7 Flowchart

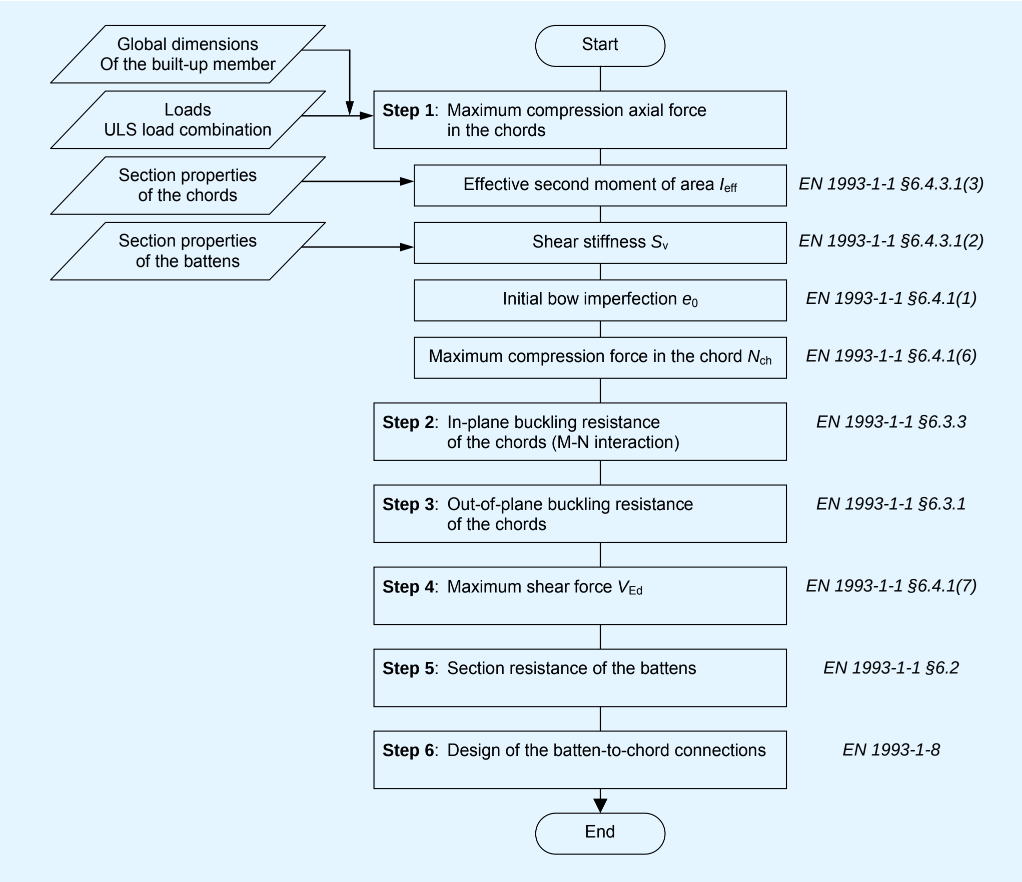


Figure 3.4 Flowchart of the design methodology for battened built-up columns

3.4 Buckling length

3.4.1 Laced compression members

Chords

According to EN 1993-1-1 Annex BB, the buckling length L_{cr} of a rolled I or H section chord member of built-up columns is taken as 0,9L for in-plane buckling and 1,0L for out-of-plane buckling. These values may be reduced if it is justified through detailed analysis.

L is the distance in a given plane between two adjacent points at which a member is braced against displacement in this plane, or between one such point and the end of the member.

Web members

Angles are mostly used as web members.

Provided that the chords supply appropriate end restraint to web members in compression made of angles and the end connections supply appropriate fixity (at least 2 bolts if bolted), the buckling length L_{cr} for in-plane buckling is taken as 0,9L, where L is the system length between joints.

When only one bolt is used for end connections of angle web members, the eccentricity should be taken into account and the buckling length L_{cr} is taken equal to the system length L.

The effective slenderness ratio $\bar{\lambda}_{eff}$ of angle web members is given in EN 1993-1-1 § BB.1.2 as follows:

$$\bar{\lambda}_{eff} = 0{,}35 + 0{,}7\bar{\lambda}$$

where:

$\bar{\lambda}$ is the non-dimensional slenderness defined in EN 1993-1-1 § 6.3.

For sections other than angles, the web members may be designed for in-plane buckling using a buckling length smaller than the system length and using the non-dimensional slenderness as defined in EN 1993-1-1 § 6.3, provided that the chords provide appropriate end restraint and the end connections provide appropriate fixity (at least 2 bolts if bolted). In practice, the buckling length L_{cr} of a rolled profile is equal to the distance between joints for in-plane buckling and for out-of-plane buckling.

3.4.2 Battened compression members

For simplicity, any potential restraint at the ends of the columns is neglected and the buckling length of the chords may be taken as the system length.

REFERENCES

1 EN 1993-1-1:2005 Eurocode 3 Design of Steel structures. General rules and rules for buildings

2 EN 1993-1-8:2005 Eurocode 3 Design of Steel structures. Design of joints

APPENDIX A

Worked Example: Design of a laced built-up column

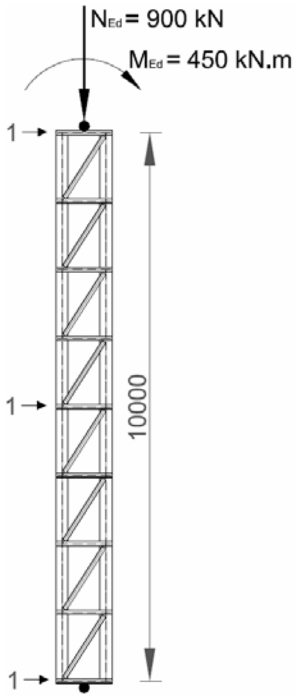
1. Introduction

This worked example deals with the verification of a typical built-up column under compressive axial force and bending moment. The calculations are carried out according to EN 1993-1-1. No National Annex is considered and the recommended values of EN 1993-1-1 are used in the calculations.

The calculations are performed according to the design methodology given in Section 3.2 of this guide.

2. Description

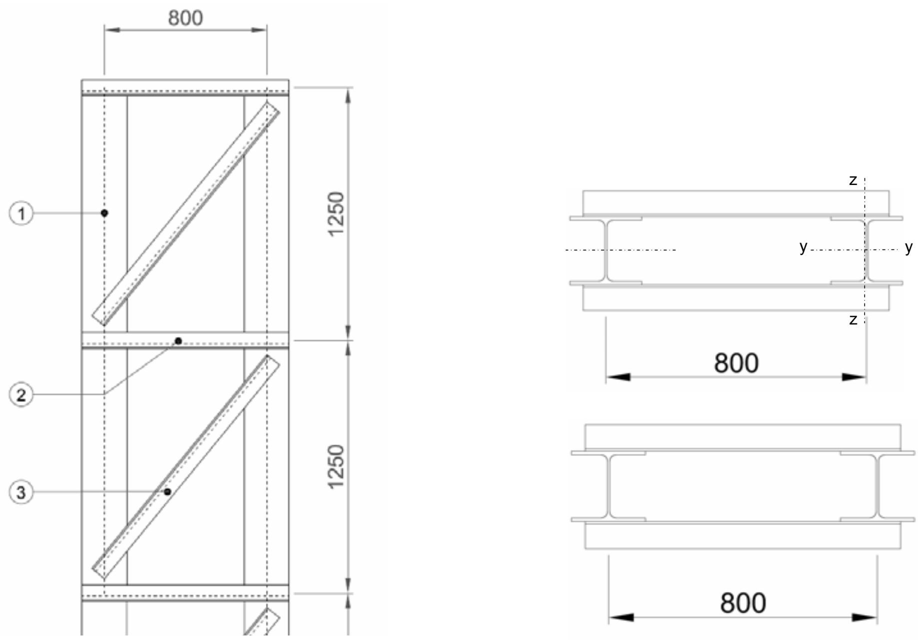
The geometry of the built-up column is described in Figure A.1 and in Figure A.2. For the most unfavourable ULS combination of actions, an axial force and a bending moment about the strong axis of the compound section are applied at the top of the column.



1 Lateral restraints

Figure A.1 Design model

The built-up column is restrained against out-of-plane buckling at both ends and at mid-height.



1 Chords HEA 200
2 Posts Angles 90 × 9
3 Diagonals Angles 80 × 8

Figure A.2 Geometry of the built-up column

Section properties

Note that the y-y axis and the z-z axis refer to the strong axis and the weak axis respectively, of the cross-section of each component.

Chords: HEA 220 – S355

$A_{ch} = 64{,}3 \text{ cm}^2$

$i_y = 9{,}17 \text{ cm} \qquad i_z = 5{,}51 \text{ cm}$

Diagonals: Equal angles L 90 × 90 × 9 – S355

$A_d = 15{,}52 \text{ cm}^2$

$i_y = i_z = 2{,}73 \text{ cm} \qquad i_u = 3{,}44 \text{ cm} \qquad i_v = 1{,}75 \text{ cm}$

Posts: Equal angles L 80 × 80 × 8 – S355

$A_v = 12{,}27 \text{ cm}^2$

$i_y = i_z = 2{,}43 \text{ cm} \qquad i_u = 3{,}06 \text{ cm} \qquad i_v = 1{,}56 \text{ cm}$

3. Step 1: Maximum compressive axial force in the chords

3.1. Effective second moment of area

The effective second moment of area of the built-up section about the strong axis is calculated using the following expression:

EN 1993-1-1 § 6.4.2.1

$I_{eff} = 0{,}5\ h_0^2 A_{ch}$

where:

A_{ch} is the section area of a chord

h_0 is the distance between the centroids of the chords

The value of the effective second moment of area is:

$I_{eff} = 0{,}5 \times 80^2 \times 64{,}3 = 205800\ \text{cm}^4$

3.2. Shear stiffness

For N-shaped arrangement of lacings, the expression of shear stiffness is:

EN 1993-1-1 Figure 6.9

$$S_v = \frac{nEA_d a h_0^2}{d^3 \left[1 + \dfrac{A_d h_0^3}{A_v d^3}\right]}$$

where:

$d \quad = \sqrt{h_0^2 + a^2} = \sqrt{0{,}8^2 + 1{,}25^2} = 1{,}48$ m

n is the number of planes of lacings ($n = 2$)

A_d is the section area of the diagonals

A_v is the section area of the posts.

Therefore:

$$S_v = \frac{2 \times 210000 \times 1552 \times 1250 \times 800^2}{1480^3 \left[1 + \dfrac{1552 \times 800^3}{1227 \times 1480^3}\right]} \times 10^{-3}$$

$S_v = 134100$ kN

3.3. Initial bow imperfection

The initial bow imperfection is taken equal to:

$e_0 = L/500 = 10000/500 = 20$ mm

EN 1993-1-1 § 6.4.1(1)

3.4. Maximum axial compressive force in the chords

The maximum compressive axial force in the chords, $N_{\mathrm{ch,Ed}}$, is determined at mid height of the built-up column as follows:

$$N_{\mathrm{ch,Ed}} = \frac{N_{\mathrm{Ed}}}{2} + \frac{M_{\mathrm{Ed}} h_0 A_{\mathrm{ch}}}{2 I_{\mathrm{eff}}}$$

EN 1993-1-1 § 6.4.1(6)

where:

$$M_{\mathrm{Ed}} = \frac{N_{\mathrm{Ed}} e_0 + M_{\mathrm{Ed}}^{\mathrm{I}}}{1 - \frac{N_{\mathrm{Ed}}}{N_{\mathrm{cr}}} - \frac{N_{\mathrm{Ed}}}{S_{\mathrm{v}}}}$$

N_{cr} is the effective critical axial force of the built up member:

$$N_{\mathrm{cr}} = \frac{\pi^2 E I_{\mathrm{eff}}}{L^2} = \frac{\pi^2 \times 210000 \times 205800 \times 10^4}{10000^2} \times 10^{-3} = 42650 \text{ kN}$$

The maximum bending moment, including the bow imperfection and the second order effects is:

$$M_{\mathrm{Ed}} = \frac{900 \times 0{,}02 + 450}{1 - \frac{900}{42650} - \frac{900}{134100}} = 481{,}4 \text{ kNm}$$

In the most compressed chord, the axial force is:

$$N_{\mathrm{ch,Ed}} = \frac{900}{2} + \frac{481{,}4 \times 0{,}8 \times 64{,}34 \times 10^{-4}}{2 \times 205800 \times 10^{-8}} = 1052 \text{ kN}$$

4. Step 2: In-plane buckling resistance of the chord

4.1. Classification of the cross-section of the chord

ε = 0,81 for steel grade S355

Flange slenderness: $c/t_{\mathrm{f}} = 88{,}5 / 11 = 8{,}05 < 10\ \varepsilon = 8{,}10$ Class 2

Web slenderness: $c/t_{\mathrm{w}} = 152 / 7 = 21{,}7 < 33\ \varepsilon = 26{,}73$ Class 1

Therefore the cross-section is Class 2 for pure compression.

4.2. Buckling resistance of a chord

The buckling resistance of the most compressed chord is verifed according to EN 1993-1-1 § 6.3.1 for buckling about the weak axis of the cross-section, i.e. about the z-z axis.

The buckling length of a hot-rolled H-section member can be taken equal to 0,9 *a* for in-plane buckling, where *a* is the system length between two nodes of the built-up column.

Buckling length of chords:

$L_{cr,z} = 0,9\ a = 0,9 \times 1,25 = 1,125$ m

EN 1993-1-1 BB.1.1(2)B

The slenderness is:

$$\lambda_z = \frac{L_{cr,z}}{i_z}$$

where

i_z is the radius of gyration of the gross cross-section, about the weak axis.

therefore: $\lambda_z = \dfrac{1125}{55,1} = 20,42$

$$\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93,9\,\varepsilon \qquad \text{With: } \varepsilon = 0,81 \text{ for steel grade S355}$$

$$\lambda_1 = 93,9 \times 0,81 = 76,06$$

The non-dimensional slenderness is:

$$\bar{\lambda}_z = \frac{\lambda_z}{\lambda_1} = \frac{20,42}{76,06} = 0,268$$

Buckling curve c for buckling about the weak axis, since:

EN 1993-1-1 Table 6.2

Steel grade S355

$h/b < 1,2$

$t_f < 100$ mm

The imperfection factor is: $\alpha_z = 0,49$

The reduction factor χ_z can be calculated from the following expressions:

EN 1993-1-1 § 6.3.1.2(1)

$$\phi_z = 0,5\left[1 + \alpha_z\left(\bar{\lambda}_z - 0,2\right) + \bar{\lambda}_z^2\right] = 0,5\left[1 + 0,49 \times \left(0,268 - 0,2\right) + 0,268^2\right] = 0,553$$

$$\chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 + \bar{\lambda}_z^2}} = \frac{1}{0,553 + \sqrt{0,553^2 - 0,268^2}} = 0,965$$

The design buckling resistance is equal to:

$$N_{b,z,Rd} = \frac{\chi_z A_{ch} f_y}{\gamma_{M1}} = \frac{0,965 \times 6430 \times 355}{1,0} \times 10^{-3} = 2203\ \text{kN}$$

The resistance criterion is:

$$\frac{N_{ch,Ed}}{N_{b,z,Rd}} = \frac{1052}{2203} = 0,477 < 1 \qquad \text{OK}$$

5. Step 3: Out-of-plane buckling resistance of the chords

The built-up column is pinned at both ends and is laterally supported at mid-height. Therefore the buckling length for buckling about the strong axis of the chords is taken equal to:

$L_{cr,y} = L/2 = 10000/2 = 5000$ mm

The slenderness is:

$$\lambda_y = \frac{L_{cr,y}}{i_y}$$

where

i_y is the radius of gyration of the gross cross-section, about the strong axis.

Therefore:

$$\lambda_y = \frac{L_{cr,y}}{i_y} = \frac{5000}{91,7} = 54,53$$

$$\lambda_1 = 93,9\,\varepsilon = 76,06$$

The non-dimensional slenderness is:

$$\bar{\lambda}_y = \frac{\lambda_y}{\lambda_1} = \frac{54,53}{76,06} = 0,717$$

Buckling curve *b* for buckling about the strong axis, since:

Steel grade S355

$h/b < 1,2$

$t_f < 100$ mm

The imperfection factor is: $\alpha_y = 0,34$

The reduction factor χ_y can be calculated from the following expressions:

EN 1993-1-1 § 6.3.1.2(1)

$$\phi_y = 0,5\left[1 + \alpha_y\left(\bar{\lambda}_y - 0,2\right) + \bar{\lambda}_y^2\right] = 0,5\left[1 + 0,34 \times \left(0,717 - 0,2\right) + 0,717^2\right] = 0,845$$

$$\chi_y = \frac{1}{\phi_y + \sqrt{\phi_y^{\,2} + \bar{\lambda}_y^2}} = \frac{1}{0,845 + \sqrt{0,845^2 - 0,717^2}} = 0,774$$

The design buckling resistance is equal to:

$$N_{b,y,Rd} = \frac{\chi_y A_{ch} f_y}{\gamma_{M1}} = \frac{0,774 \times 6430 \times 355}{1,0} \times 10^{-3} = 1767\text{ kN}$$

The resistance criterion is:

$$\frac{N_{ch,Ed}}{N_{b,y,Rd}} = \frac{1052}{1767} = 0,595 < 1 \quad \text{OK}$$

6. Step 4: Maximum shear force

The maximum compressive axial force is obtained in the diagonals of the end panels of the built-up column. It depends on the shear force in this panel. The shear force can be assessed by the following expression:

$$V_{\mathrm{Ed}} = \frac{1}{L}\left(4-(4-\pi)\frac{e_{\mathrm{o}}N_{\mathrm{Ed}}}{e_{\mathrm{o}}N_{\mathrm{Ed}}+M_{\mathrm{Ed}}^{I}}\right)M_{\mathrm{Ed}}^{II}$$

where:

L = 10 m

e_0 = 0,02 m

N_{Ed} = 900 kN

$M_{\mathrm{Ed}}^{\mathrm{I}}$ = 450 kNm

$M_{\mathrm{Ed}}^{\mathrm{II}}$ = 482 kNm

Therefore:

$$V_{\mathrm{Ed}} = \frac{1}{10}\left(4-(4-\pi)\frac{0{,}02\times 900}{0{,}02\times 900+450}\right)\times 482 = 191{,}2\ \mathrm{kN}$$

7. Step 5: Buckling resistance of the web members in compressive

7.1. Diagonals

7.1.1. Maximum compression axial force

The expression of the compression axial force $N_{\mathrm{d,Ed}}$ in a diagonal is derived from the shear force as follows:

$$N_{\mathrm{d,Ed}} = \frac{V_{\mathrm{Ed}}\cos\varphi}{n} = \frac{V_{\mathrm{Ed}}d}{nh_0}$$

where:

h_0 = 800 mm

d = 1480 mm

n is the number of plans of lacings: $n = 2$

then:

$$N_{\mathrm{d,Ed}} = \frac{191{,}2\times 1480}{2\times 800} = 176{,}86\ \mathrm{kN}$$

7.1.2. Classification of a diagonal in compression

$h/t = 90 / 9 = 10 \quad < 15\,\varepsilon = 12{,}15$

$(b+h) / (2t) = (90+90) / (2 \times 9) = 10 \quad > 11{,}5\,\varepsilon = 9{,}31$ Class 4

EN 1993-1-1 Table 5.2 Sheet 3

Although the cross-section is Class 4, according to EN 1993-1-1 Table 5.2 Sheet 3, the calculation of the effective section area leads to no reduction. The section area is therefore fully effective and the calculation is the same as for a Class 3 Section.

7.1.3. Buckling resistance of a diagonal

The non dimensional slenderness can be calculated according to EN 1993-1-1 § BB.1.2 in so far as the diagonals are welded at both ends and the chords are stiff enough to ensure that the ends are clamped.

Slenderness about the weakest axis:

$$\lambda_{\mathrm{v}} = \frac{d}{i_{\mathrm{v}}} = \frac{1480}{17{,}5} = 84{,}57$$

Non dimensional slenderness

$$\bar{\lambda}_{\mathrm{v}} = \frac{\lambda}{93{,}9\varepsilon} = \frac{84{,}57}{93{,}9 \times 0{,}81} = 1{,}112$$

Effective non dimensional slenderness

EN 1993-1-1 § BB.1.2

$$\bar{\lambda}_{\mathrm{eff,v}} = 0{,}35 + 0{,}7\bar{\lambda}_{\mathrm{v}} = 0{,}35 + 0{,}7 \times 1{,}112 = 1{,}128$$

Buckling curve *b* is used for the determination of the reduction factor:

$\alpha_{\mathrm{v}} = 0{,}34$

Therefore:

EN 1993-1-1 § 6.3.1

$$\phi_{\mathrm{v}} = 0{,}5\left[1 + \alpha\left(\bar{\lambda}_{\mathrm{eff,v}} - 0{,}2\right) + \bar{\lambda}_{\mathrm{eff,v}}^2\right] = 0{,}5 \times \left[1 + 0{,}34 \times (1{,}128 - 0{,}2) + 1{,}128^2\right] = 1{,}294$$

$$\chi_{\mathrm{v}} = \frac{1}{\phi_{\mathrm{v}} + \sqrt{\phi_{\mathrm{v}}^2 + \bar{\lambda}_{\mathrm{eff,v}}^2}} = \frac{1}{1{,}294 + \sqrt{1{,}294^2 - 1{,}128^2}} = 0{,}519$$

The design buckling resistance of a compression member is equal to:

$$N_{\mathrm{b\text{-}d,Rd}} = \frac{\chi_{\mathrm{v}} A_{\mathrm{d}} f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}} = \frac{0{,}519 \times 1552 \times 355}{1{,}0} \times 10^{-3} = 285{,}9\ \mathrm{kN}$$

The resistance criterion is:

$$\frac{N_{\mathrm{d,Ed}}}{N_{\mathrm{b\text{-}d,Rd}}} \leq 1 \Leftrightarrow \frac{176{,}8}{285{,}9} = 0{,}62 < 1 \qquad \text{OK}$$

7.2. Posts

7.2.1. Maximum compressive axial force

The maximum compressive axial force is:

$N_{h,Ed} = V_{Ed} = 191{,}2$ kN

7.2.2. Classification of the cross-section

$h/t = 80 / 8 = 10 \quad < 15\,\varepsilon = 12{,}15$

$(b+h) / (2t) = (80+80) / (2 \times 8) = 10 \quad > 11{,}5\,\varepsilon = 9{,}31$ Class 4

EN 1993-1-1 Table 5.2 Sheet 3

Although the cross-section is Class 4, according to EN 1993-1-1 Table 5.2 Sheet 3, the calculation of the effective section area leads to no reduction. The section area is therefore fully effective and the calculation is the same as for a Class 3 section.

7.2.3. Buckling resistance

The buckling length is equal to:

$L_{cr} = h_0 = 800$ mm

Slenderness about the weakest axis:

$$\lambda_v = \frac{L_{h,y}}{i_v} = \frac{800}{15{,}6} = 51{,}28$$

Non dimensional slenderness:

$$\bar{\lambda}_v = \frac{\lambda_v}{93{,}9\varepsilon} = \frac{51{,}28}{93{,}9 \times 0{,}81} = 0{,}674$$

Effective non dimensional slenderness:

EN 1993-1-1 § BB.1.2

$$\bar{\lambda}_{eff,v} = 0{,}35 + 0{,}7\bar{\lambda}_v = 0{,}35 + 0{,}7 \times 0{,}674 = 0{,}822$$

The buckling curve **b** is used for the determination of the reduction factor:

$\alpha = 0{,}34$

Therefore:

$$\phi_v = 0{,}5\left[1 + \alpha\left(\bar{\lambda}_{eff,v} - 0{,}2\right) + \bar{\lambda}^2_{eff,v}\right] = 0{,}5 \times \left[1 + 0{,}34 \times (0{,}822 - 0{,}2) + 0{,}822^2\right] = 0{,}943$$

$$\chi_v = \frac{1}{\phi_v + \sqrt{\phi_v^2 + \bar{\lambda}^2_{eff,v}}} = \frac{1}{0{,}943 + \sqrt{0{,}943^2 - 0{,}822^2}} = 0{,}712$$

The design buckling resistance of a compression member is equal to:

$$N_{b,Rd} = \frac{\chi_v A_h f_y}{\gamma_{M1}} = \frac{0{,}712 \times 1227 \times 355}{1{,}0} \times 10^{-3} = 310 \text{ kN}$$

The resistance criterion is:

$$\frac{N_{\mathrm{h,Ed}}}{N_{\mathrm{b,Rd}}} = \frac{191{,}2}{310} = 0{,}62 < 1 \qquad \text{OK}$$

8. Step 6: Resistance of the web members in tension

It is necessary to verify the resistance of the diagonals in tension, even if this situation is generally less critical than compression.

The verification of these members includes the verification of the resistance of the cross-section and the verification of the resistance of the net section for bolted connections.

Maximum design value of the tensile axial force:

$N_{\mathrm{t,Ed}} = 176{,}8$ kN

The resistance criterion is:

EN 1993-1-1 §6.2.3

$$\frac{N_{\mathrm{t,Ed}}}{N_{\mathrm{t,Rd}}} \leq 1{,}0$$

The design tension resistance $N_{\mathrm{t,Rd}}$ is taken as the design plastic resistance of the gross cross-section:

$$N_{\mathrm{t,Rd}} = N_{\mathrm{pl,Rd}} = \frac{A_{\mathrm{d}}\, f_{\mathrm{y}}}{\gamma_{\mathrm{M}_0}} = \frac{1552 \times 355}{1{,}0} \times 10^{-3} = 551\,\mathrm{kN}$$

The resistance criterion is:

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{t,Rd}}} = \frac{176{,}8}{551{,}0} = 0{,}32 < 1{,}0 \quad \text{OK}$$

9. Step 7: Resistance of the diagonal-to-chord welded connection

The diagonals (L90 × 90 × 9) are welded to the chord (HEA 220) by fillet welds, see Figure A.3.

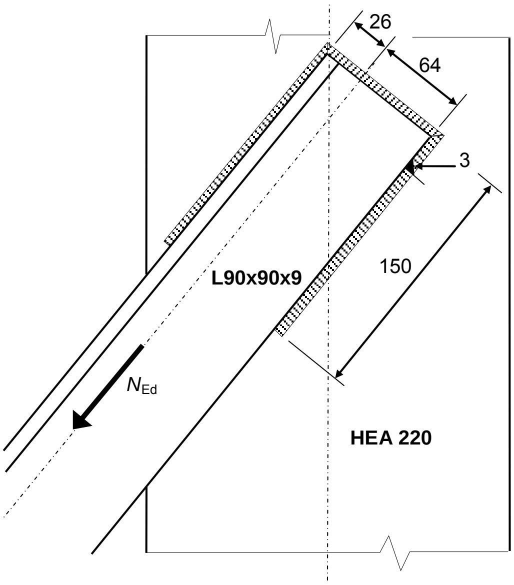


Figure A.3 Welded connection of a diagonal to the chord

Throat thickness:	a	= 3 mm
Effective longitudinal length of the fillet weld:	$l_{\text{eff-L}}$	= 150 mm
Effective transverse length of the fillet weld:	$l_{\text{eff-T}}$	= 90 mm
Axial force in the diagonal:	$N_{\text{d,Ed}}$	= 176,8 kN

The design resistance of a fillet weld is determined using the simplified method given in EN 1993-1-8 § 4.5.3.3.

At every point along the length of the fillet weld, the resultant of all the forces per unit length transmitted by the weld should satisfy the following criterion:

$$F_{\text{w,Ed}} \leq F_{\text{w,Rd}}$$

where:

$F_{\text{w,Ed}}$ is the design value of the force per unit length

$F_{\text{w,Rd}}$ is the design weld resistance per unit length

The design resistance is independent of the orientation of the weld throat plane and it is determined from:

$$F_{\text{w,Rd}} = f_{\text{vw,d}}\, a$$

where: EN1993-1-8

$f_{\text{vw,d}}$ is the design shear strength of the weld § 4.5.3.3

$$f_{\text{vw,d}} = \frac{f_{\text{u}} / \sqrt{3}}{\beta_{\text{w}} \gamma_{\text{M2}}}$$

f_u is the ultimate tensile strength of the weaker part:

$f_u = 510$ N/mm^2

EN 1993-1-1 Table 3.1

β_w is the appropriate correlation factor:

$\beta_w = 0{,}9$ for steel grade S355

EN1993-1-8 Table 4.1

$\gamma_{M2} = 1{,}25$

therefore:

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}} = \frac{510/\sqrt{3}}{0{,}9 \times 1{,}25} = 261{,}7 \text{ N/mm}^2$$

$$F_{w,Rd} = f_{vw,d} a = 261{,}7 \times 5 = 785{,}2 \text{ N/mm}$$

$$F_{w,Ed} = \frac{N_{d,Ed}}{\sum l_{eff}} = \frac{176800}{(2 \times 150 + 90)} = 453{,}3 \text{ N/mm}$$

Therefore:

$F_{w,Ed} = 453{,}3$ N/mm^2 $< F_{w,Rd} = 785{,}2$ N/mm^2 OK

The minimum throat thickness $a_{min} = 3$ mm is acceptable.

To prevent corrosion, the diagonal may be welded all around in one pass ($a = 3$ mm).

To account for eccentricity a 5 mm (2 passes) throat fillet weld is recommended on the unconnected leg side, as shown in Figure A.4.

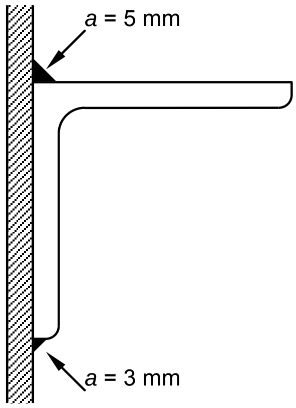


Figure A.4 Throat thickness of the weld fillets