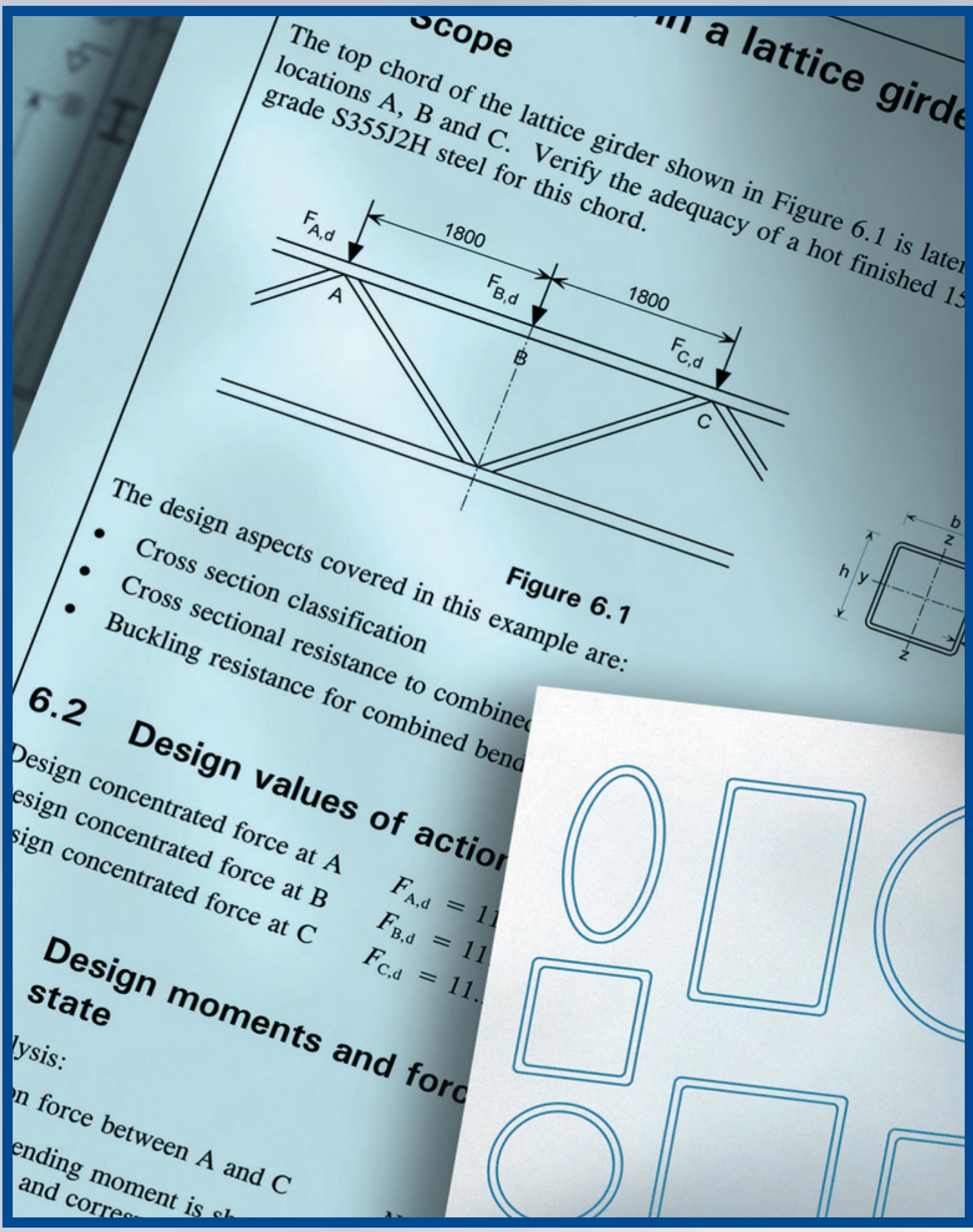
UK EUROCODES



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Steel Building Design:

Worked Examples - Hollow Sections

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The Steel Construction Institute, Silwood Park, Ascot, Berkshire, SL5 7QN.
Telephone: +44 (0) 1344 636525 Fax: +44 (0) 1344 636570
Email: membership@steel-sci.com

For information on publications, telephone direct: +44 (0) 1344 636513
or Email: publications@steel-sci.com

For information on courses, telephone direct: +44 (0) 1344 636500
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Steel Building Design:
Worked Examples - Hollow Sections

In accordance with Eurocodes and the UK National Annexes

M E Brettle BEng (Hons)

Published by:
The Steel Construction Institute
Silwood Park
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Berkshire SL5 7QN

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Fax: 01344 636570

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FOREWORD

The design of steel framed buildings in the UK has, since 1990, generally been in accordance with the British Standard BS 5950-1. However, that Standard is due to be withdrawn in March 2010; it will be replaced by the corresponding Parts of the Structural Eurocodes.

The Eurocodes are a set of structural design standards, developed by CEN (European Committee for Standardisation) over the last 30 years, to cover the design of all types of structures in steel, concrete, timber, masonry and aluminium. In the UK, they are published by BSI under the designations BS EN 1990 to BS EN 1999; each of these ten Eurocodes is published in several Parts and each Part is accompanied by a National Annex that implements the CEN document and adds certain UK-specific provisions.

This publication is one of a number of new design guides that are being produced by SCI to help designers become acquainted with the use of the Eurocodes for structural steel design. It provides a number of short examples, in the form of calculation sheets, illustrating the design of structural hollow sections for beams and columns in buildings.

All the examples were prepared by Miss M E Brettle and checked by Mr A S Malik of The Steel Construction Institute.

The work leading to this publication was funded by Tata Steel* and their support is gratefully acknowledged.

* This publication includes references to Corus, which is a former name of Tata Steel in Europe

Contents

SUMMARY

This publication presents seven design examples to illustrate the use of Eurocode 3 for the design of structural hollow section members. The examples all use the Nationally Determined Parameter values recommended in the UK National Annexes.

A brief introductory section precedes the examples and a bibliography section is given at the end.

1 INTRODUCTION

1.1 Scope

This publication provides seven worked examples illustrating the design of members in buildings. All the members in these examples are hot finished structural hollow sections.

The examples illustrate the verification of the members in accordance with Eurocode 3, as implemented by the UK National Annexes to its various Parts. References are mainly to Part 1-1 (BS EN 1993-1-1) but some aspects are verified in accordance with Part 1-8 (BS EN 1993-1-8). Reference is also made to BS EN 1990. This publication should be used in conjunction with the Eurocodes themselves and other relevant SCI publications, in particular, *Steel building design: Introduction to the Eurocodes* (P361) and *Steel building design: Design data* (P363).

1.2 Basis of structural design

EN 1990 *Eurocode – Basis of structural design* sets out the principles that apply to structural design according to the Eurocodes. It is used in conjunction with the material-specific Eurocodes, notably, for the present publication, with EN 1993 *Eurocode 3 Design of steel structures*.

EN 1990 sets out a limit state design basis, gives rules for determining design values of actions and combinations of actions, and states the verifications that are required at ultimate and serviceability limit states.

1.2.1 National Choice

Each country in Europe may publish the main body of a Eurocode Part with an accompanying National Annex[†]. The principles and application rules given within the main body of a Eurocode Part do not differ between countries. However, within the main body there are some provisions for national choice to be exercised in the selection of design method and in the setting of values of parameters (collectively known as Nationally Determined Parameters, NDPs). Most notably, the partial factors applied to actions and to resistances may be set by the country. The exercise of these national choices and the setting of NDPs is given in the National Annex that accompanies the Eurocode Part.

The worked examples in this publication use the NDPs recommended in the UK National Annexes to the Eurocode Parts.

In general, the National Annex for the country where the structure is to be constructed should always be consulted in the design of a structure.

[†] Note that the main body of all the Eurocode Parts is issued initially by CEN as an 'EN' document - for example EN 1990. The main body is then issued in each country by the national standards organisation, for example, in UK by BSI, as BS EN 1990. The National Annex may be part of that document or may be issued separately.

1.2.2 Verification at ultimate limit state

For verification of persistent and transient situations at ULS, EN 1990 gives the alternative of two methods to determine the design value of the effects of combined actions. The design value may be determined from either expression (6.10) or from expressions (6.10a) and (6.10b).

The first method is to express the combination of actions as:

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j} + \gamma_P P + \gamma_{Q,1}Q_{k,1} + \sum_{i\geq 1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \tag{6.10}$$

The second method is to use the less favourable value determined from the following two expressions:

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j} + \gamma_P P + \gamma_{Q,1}\psi_{0,1}Q_{k,1} + \sum_{i\geq 1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \tag{6.10a}$$

$$\sum_{j\geq 1}\xi_j\gamma_{G,j}G_{k,j} + \gamma_P P + \gamma_{Q,1}Q_{k,1} + \sum_{i\geq 1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \tag{6.10b}$$

where:

$G_{k,j}$ unfavourable permanent action

P prestressing action

$Q_{k,1}$ leading variable action

$Q_{k,i}$ accompanying variable actions ($i > 1$)

γ, ψ and ξ are partial, combination and reduction factors.

Table A1.2(B) in Annex A of EN 1990 presents the three expressions along with the recommended values for the partial and reduction factors. The recommended values for the combination factors are given in Table A1.1 of EN 1990.

The National Annex for the country in which the building is to be constructed should be consulted for guidance on which of the two methods to use and the values to use for the factors. The UK National Annex allows the use of either method and adopts the factor values recommended in the main text of EN 1990. (It is known that some countries only adopt the first method.)

The first method is the simplest to apply, as only one expression is used. However, it has been found that for the majority of situations, a lower design value of the effect of combined actions may be obtained by the use of the second method (expressions (6.10a) and (6.10b)) and, in the UK, expression (6.10b) will in most cases be the more onerous of these two.

The worked examples in this publication use the second method to determine the design value of actions for the ultimate limit state.

1.2.3 Verification at serviceability limit state

For verification at SLS, EN 1990 gives expressions for combinations of actions at reversible and irreversible limit states. The only SLS verifications considered in this publication relate to the deflection of beams. No SLS limits are given in the Eurocode and the UK National Annex only quotes suggested limits. These combinations and suggested limits are shown where relevant.

1.3 Actions on structures

The various Parts of EN 1991 set out the characteristic values of all the different types of actions (i.e. imposed loads and imposed deformations) that structures may be subjected to. There is a distinction between permanent actions and variable actions.

In this publication, values for actions are simply stated, rather than taken explicitly from EN 1991; only vertical forces due to permanent actions (dead load) and variable actions (imposed loads) are considered.

1.4 Design of structural steelwork

For the design of structural steelwork using structural hollow sections, the following information should be noted.

1.4.1 Steel material properties

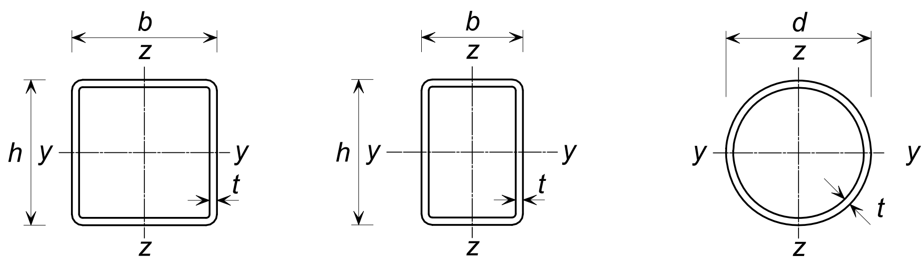
The steel grade of the structural hollow sections considered in this publication is S355J2H in accordance with EN 10210-1 and S355JR for the Tee stubs in accordance with EN 10025-2.

1.4.2 Section properties and dimensions

The reference standard for the dimensions of hot finished hollow sections is *EN 10210 – Hot finished hollow sections of non-alloy and fine grain structural steel.* Section properties have been taken from publication P363 (see Bibliography).

1.4.3 Axis Notation

For structural members the axis notation used in the Eurocodes and this publication is:



Major axis *y-y*
Minor axis *z-z*
Longitudinal axis along the member *x-x*

Figure 1.1 *Axis convention and symbols for principal dimensions*

1.5 Non contradictory complementary information (NCCI)

The application rules in the Eurocodes do not cover every aspect of design and reference must in some cases be made to additional information (such as expressions to determine elastic critical buckling values), published elsewhere. Such information is referred to as non contradictory complementary information (NCCI). NCCI also provides additional guidance that will assist the designer when designing a structure to the Eurocodes. The National Annexes may give references to NCCI documents.

Where an NCCI document has been used in this publication a reference is given. Examples of NCCI documents are those available on the Access Steel website: www.access-steel.com.

2 WORKED EXAMPLES

Silwood Park, Ascot, Berks SL5 7QN
Telephone: (01344) 636525
Fax: (01344) 636570

1 Tension member and tee connection

References are to BS EN 1993-1-1: 2005, including its National Annex, unless otherwise stated.

1.1 Scope

Verify the adequacy of the internal steelwork tension member and tee connection shown in Figure 1.1.

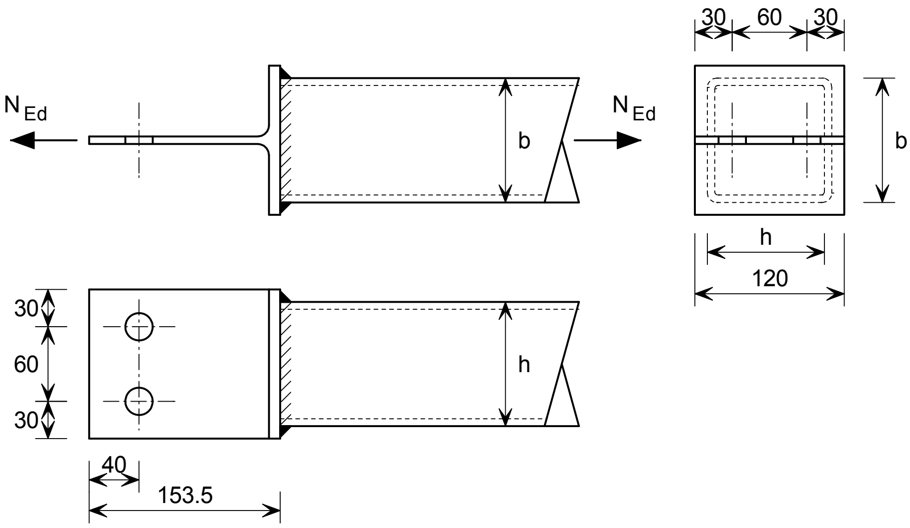


Figure 1.1

The design aspects covered in this example are:

- Cross-sectional resistance to axial tension
- Tension resistance of the SHS at the connection
- Tension resistance of Tee-stub web
- Resistance of a group of bolts
- Resistance of fillet welds
- Selection of steel sub-grade for the SHS

1.2 Design force for ultimate limit state

Design tension force N_{Ed} = 140 kN.

1.3 Section properties

Hot finished 100 × 100 × 6.3 SHS in S355 steel — P363

Depth of section h = 100 mm
Width of section b = 100 mm
Wall thickness t = 6.3 mm
Cross-sectional area A = 23.20 cm^2

For buildings that will be built in the UK the nominal values of the yield strength (f_y) and the ultimate strength (f_u) for structural steel should be those obtained from the product standard. Where a range is given the lowest nominal value should be used. — NA.2.4

For S355 steel — BS EN 10210-1 Table A.3
Yield strength ($t \leq 16$ mm) $f_y = R_{eH} = 355$ N/mm^2
Ultimate tensile strength (3 mm $\leq t \leq$ 100 mm) $f_u = R_m = 470$ N/mm^2

127 × 152 × 21 UK Tee stub (UKT) cut from 305 × 127 × 42 UKB in S355 steel

Thickness of web t_w = 8.0 mm — P363
Thickness of flange t_f = 12.1 mm

For S355 steel — BS EN 10025-2 Table 7
Yield strength ($t \leq 16$ mm) $f_y = R_{eH} = 355$ N/mm^2
Ultimate tensile strength (3 mm $\leq t \leq$ 100 mm) $f_u = R_m = 470$ N/mm^2

1.4 Connection details

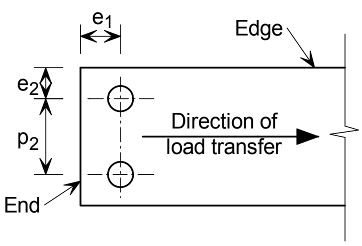


Figure 1.2

M20 Class 8.8 bolts (Class A: Bearing type connection) — BS EN 1993-1-8 3.4.1(1)
Bolt diameter d = 20 mm
Hole diameter d_0 = 22 mm
Tensile stress area of the bolt A_s = 245 mm^2 — P363, Page D-303
Yield strength of bolt f_{yb} = 640 N/mm^2 — BS EN 1993-1-8 Table 3.1
Ultimate tensile strength of bolt f_{ub} = 800 N/mm^2

Dimensions
End distance e_1 = 40 mm
Edge distance e_2 = 30 mm
Spacing p_2 = 60 mm

Dimensional limits for a connection that is not exposed to the weather or other corrosive influences — BS EN 1993-1-8 Table 3.3

$1.2d_0 \leq e_1$; $1.2 \times 22 = 26.4$ mm $<$ 40 mm

$1.2d_0 \leq e_2$; 26.4 mm $<$ 30 mm

$2.4d_0 \leq p_2 \leq \min(14t_w$ or 200 mm)

$14t_w = 14 \times 8 = 112.0$ mm $<$ 200 mm

$2.4d_0 = 2.4 \times 22 = 52.8$ mm

52.8 mm $<$ 60.0 mm $<$ 112.0 mm

Therefore the dimensions of the connection are satisfactory.

1.5 Partial factors for resistance

1.5.1 Structural Steel

$\gamma_{M0} = 1.0$ — NA.2.15

$\gamma_{M2} = 1.1$

$\gamma_{M2} = 1.25$ (plates in bearing in bolted connections) — BS EN 1993-1-8 Table NA.1

1.5.2 Bolts

$\gamma_{M2} = 1.25$ — BS EN 1993-1-8 Table NA.1

1.5.3 Welds

$\gamma_{M2} = 1.25$ — BS EN 1993-1-8 Table NA.1

1.6 Cross-sectional resistance

1.6.1 Tension resistance of SHS (whole cross section)

Verify that:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0$$ — 6.2.3(1)

The design tension resistance of the cross section is:

$$N_{t,Rd} = N_{pl,Rd} = \frac{A \times f_y}{\gamma_{M0}}$$

$$N_{t,Rd} = \frac{2320 \times 355}{1.0} \times 10^{-3} = 824 \text{ kN}$$

$$\frac{N_{Ed}}{N_{t,Rd}} = \frac{140}{824} = 0.17 < 1.0$$ — 6.2.3(1)

Therefore the tension resistance of the SHS cross section is adequate.

1.6.2 Tension resistance of SHS at welded connection

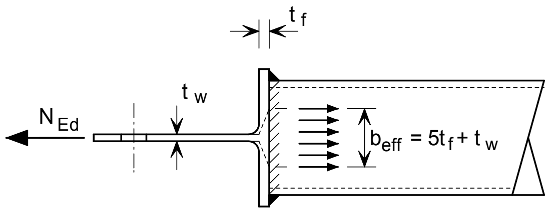


Figure 1.3

The method given below is based on that given in *Hollow structural section, connections and trusses. A design guide (2nd edition)* by J A Parker and J E Henderson, 1997.

Assuming a load distribution of 2.5 : 1 through the flange the effective width (b_{eff}) is:

$$b_{eff} = 5t_f + t_w \quad \text{but, } b_{eff} \le b$$

$$b_{eff} = 5t_f + t_w = (5 \times 12.1) + 8 = 68.5 \text{ mm} < b = 100 \text{ mm}$$

Note: The root radius of the tee-stub has been ignored which is a conservative approach.

The effective tension resistance of the SHS at the connection is:

$$N_{t,Rd} = \frac{2b_{eff} f_y t}{\gamma_{M0}}$$

$$N_{t,Rd} = \frac{2 \times 68.5 \times 355 \times 6.3}{1.0} \times 10^{-3} = 306 \text{ kN}$$

$$\frac{N_{Ed}}{N_{t,Rd}} = \frac{140}{306} = 0.46 < 1.0$$

Therefore the tension resistance of the SHS at the connection is adequate.

1.6.3 Tension resistance of the Tee-stub web

Three failure surfaces should be considered when determining the tension resistance of the tee stub web.

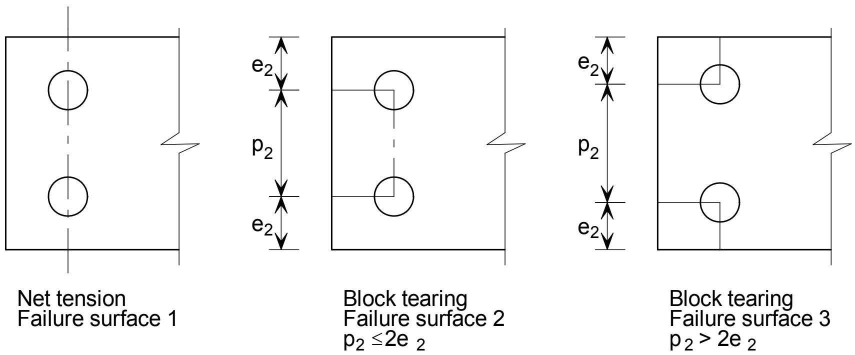


Figure 1.4

Net tension – Failure surface 1 — 6.2.3(1)

Verify that:

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{t,Rd}}} \leq 1.0$$

For a cross section with holes, the design tension resistance is taken as the smaller of $N_{\mathrm{pl,Rd}}$ and $N_{\mathrm{u,Rd}}$: — 6.2.3(2)

a) $$N_{\mathrm{pl,Rd}} = \frac{A \times f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} \qquad (6.6)$$

$$A = A_{\mathrm{web}} = 120 t_{\mathrm{w}} = 120 \times 8 = 960 \ \mathrm{mm}^2$$

$$N_{\mathrm{pl,Rd}} = \frac{960 \times 355}{1.0} \times 10^{-3} = 341 \ \mathrm{kN}$$

b) $$N_{\mathrm{u,Rd}} = \frac{0.9 \times A_{\mathrm{net}} \times f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}} \qquad (6.7)$$

$$A_{\mathrm{net}} = A_{\mathrm{web}} - 2 d_0 t_{\mathrm{w}} = 960 - (2 \times 22 \times 8) = 608 \ \mathrm{mm}^2$$ — 6.2.2.2

$$N_{\mathrm{u,Rd}} = \frac{0.9 \times 608 \times 470}{1.1} \times 10^{-3} = 234 \ \mathrm{kN}$$

Since $N_{\mathrm{u.Rd}} < N_{\mathrm{pl.Rd}}$ the tension resistance of the web is:

$$N_{\mathrm{t,Rd}} = N_{\mathrm{u,Rd}} = 234 \ \mathrm{kN}$$

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{t,Rd}}} = \frac{140}{234} = 0.60 < 1.0$$ — 6.2.3(1)

Therefore, the tension resistance of the tee-stub along failure surface 1 is adequate.

Block tearing - Failure surface 2

For block tearing the most onerous case of failure surface 2 or 3 should be considered. In this example $p_2 = 2e_2$; therefore failure surface 2 is considered.

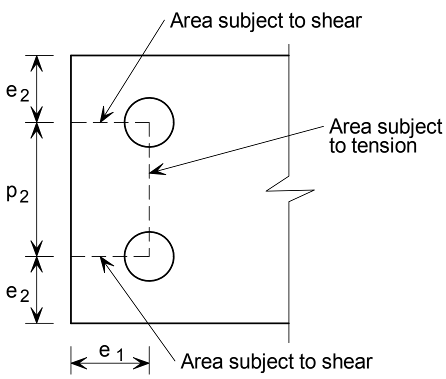


Figure 1.5

Verify that:

$$\frac{N_{\mathrm{Ed}}}{V_{\mathrm{Eff,1,Rd}}} \leq 1.0$$

For a symmetric bolt group subject to concentric loading, the design block tearing resistance ($V_{\mathrm{Eff,1,Rd}}$) is determined from:

BS EN 1993-1-8 3.10.2(2)

$$V_{\mathrm{Eff,1,Rd}} = \frac{f_u A_{nt}}{\gamma_{M2}} + \left(1/\sqrt{3}\right)\frac{f_y A_{nv}}{\gamma_{M0}}$$

where:

A_{nt} is the net area subject to tension

A_{nv} is the net area subject to shear

$$A_{nt} = (p_2 - d_0)t_w = (60 - 22) \times 8 = 304 \text{ mm}^2$$

$$A_{nv} = 2\left(e_1 - \frac{d_0}{2}\right)t_w = 2 \times \left(40 - \frac{22}{2}\right) \times 8 = 464 \text{ mm}^2$$

$$V_{\mathrm{Eff,1,Rd}} = \left[\frac{470 \times 304}{1.1} + \left(\frac{1}{\sqrt{3}}\right) \times \frac{355 \times 464}{1.0}\right] \times 10^{-3} = 225 \text{ kN}$$

$$\frac{N_{Ed}}{V_{\mathrm{Eff,1,Rd}}} = \frac{140}{225} = 0.62 < 1.0$$

Therefore, the block tearing resistance of the tee-stub along failure surface 2 is adequate.

Therefore, the tension resistance of the tee-stub web is adequate.

1.7 Resistance of the bolts

References in Section 1.7 are to BS EN 1993-1-8, including its National Annex

$$\frac{N_{Ed}}{F_{\mathrm{Rd,\,joint}}} \leq 1.0$$

$F_{\mathrm{rd,joint}}$ is the resistance of the group of bolts.

1.7.1 Design bearing resistance of a single bolt

Table 3.4

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}}$$

α_b is the smaller of α_d, $\frac{f_{ub}}{f_u}$ and 1.0.

For end bolts

$$\alpha_d = \frac{e_1}{3d_0} = \frac{40}{3 \times 22} = 0.61$$

Sheet 2

$$\frac{f_{ub}}{f_u} = \frac{800}{470} = 1.70$$

Therefore, $\alpha_b = \alpha_d = 0.61$

For edge bolts

k_1 is the smaller of $\frac{2.8e_2}{d_0} - 1.7$ or 2.5

$$\frac{2.8e_2}{d_0} - 1.7 = \frac{2.8 \times 30}{22} - 1.7 = 2.12$$

Therefore, $k_1 = 2.12$

$\gamma_{M2} = 1.25$ (For plates in bearing) — Sheet 3

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t_w}{\gamma_{M2}} = \frac{2.12 \times 0.61 \times 470 \times 20 \times 8}{1.25} \times 10^{-3} = 78 \text{ kN}$$

As the lap joint has a single row of bolts, $F_{b,Rd}$ should also be limited to:

3.6.1(10)

$$F_{b,Rd} \leq \frac{1.5 f_u d t_w}{\gamma_{M2}} = \frac{1.5 \times 470 \times 20 \times 8}{1.25} \times 10^{-3} = 90 \text{ kN}$$

78 kN < 90 kN

Therefore, the design bearing resistance of a single bolt is:

$F_{b,Rd} = 78$ kN

1.7.2 Design shear resistance of a single bolt

Table 3.4

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

For Class 8.8 bolts, assuming that the shear plane passes through the threaded portion of the bolt. $\alpha_v = 0.6$.

$A = A_s = 245$ mm^2 (tensile stress area of the bolt)

Therefore, the design shear resistance of one bolt in single shear is:

$$F_{v,Rd} = \frac{0.6 \times 800 \times 245}{1.25} \times 10^{-3} = 94 \text{ kN}$$

1.7.3 Design resistance of a group of bolts

For a single bolt $F_{b,Rd} = 78$ kN $< F_{v,Rd} = 94$ kN

Therefore the resistance of the group of 2 bolts is: — 3.7(1)

$F_{Rd,joint} = 2 \times 78 = 156$ kN

Design shear force applied to the joint is: $F_{v,Ed} = N_{Ed} = 140$ kN

$$\frac{N_{Ed}}{F_{Rd,joint}} = \frac{140}{156} = 0.90 < 1.0$$

Therefore two M20 grade 8.8 bolts are satisfactory.

In this example the bearing of bolts on the tee stub web is critical.

Note: If the gusset plate is thinner than the web of the T-stub this would be critical.

1.8 Fillet weld design

References in Section 1.8 are to BS EN 1993-1-8, including its National Annex

The simplified method for calculating the design resistance of the fillet weld is used here.

Consider a fillet weld with a 6 mm leg length (i.e. throat a = 4.2 mm).

Verify that: 4.5.3.3(1)

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

The design weld resistance per unit length, 4.5.3.3(2)

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

where:

$$f_{\mathrm{vw,d}} = \frac{f_{\mathrm{u}} / \sqrt{3}}{\beta_{\mathrm{w}} \gamma_{\mathrm{M2}}}$$ 4.5.3.3(3)

For S355 steel,

$\beta_{\mathrm{w}} = 0.9$ Table 4.1

f_{u} relates to the weaker part jointed, therefore: 4.5.3.2(6)

$f_{\mathrm{u}} = 470\ \mathrm{N/mm^2}$

$\gamma_{\mathrm{M2}} = 1.25$ Sheet 3

Hence $f_{\mathrm{vw,d}} = \dfrac{470 / \sqrt{3}}{0.9 \times 1.25} = 241\ \mathrm{N/mm^2}$ 4.5.3.3(3)

Therefore, the design weld resistance per mm is: 4.5.3.3(2)

$F_{\mathrm{w,Rd}} = 241 \times 4.2 \times 10^{-3} = 1.01\ \mathrm{kN/mm}$

Consider the length b_{eff} of the tee-stub. The design force is transferred over a length b_{eff} on two walls of the SHS. Therefore, the effective weld length is: For b_{eff} see Section 1.6.2 of this example

$l = 2b_{\mathrm{eff}} = 2 \times 68.5 = 137\ \mathrm{mm}$

The design weld force per mm is:

$$F_{\mathrm{w,Ed}} = \frac{N_{\mathrm{Ed}}}{l} = \frac{140}{137} = 1.02\ \mathrm{kN/mm}$$

$$\frac{F_{\mathrm{w,Ed}}}{F_{\mathrm{w,Rd}}} = \frac{1.01}{1.02} = 0.99$$ 4.5.3.3(1)

Therefore the design resistance of the weld with a leg length of 6 mm and throat thickness of 4.2 mm is satisfactory. Provide this fillet weld all round the SHS.

However, it should be noted that a larger value for the design resistance of the fillet weld is obtained when the more rigorous directional method is used. This method has been used to determine the resistance values given in SCI P363 (see Section 1.10.3 of this example).

1.9 Selection of steel sub-grade

Here only the steel sub-grade for the SHS is determined, in practice the sub-grade for the UKT should also be determined.

BS EN 1993-1-10 presents a table with limiting thicknesses for different steel sub-grades with different stress levels for a range of reference temperatures. Six variables are used in the expression given to determine the required reference temperature that should be considered. The UK National Annex presents a modified table for a single stress level, with an adjustment to reference temperature for actual stress level.

The UK National Annex also refers to non contradictory complimentary information (NCCI) given in Published Document PD 6695-1-10 for further guidance.

The procedure for determining the maximum thickness values for steelwork in buildings is given in 2.2 of PD 6695-1-10, with reference to Tables 2 and 3 in that document. That guidance is used in this example.

1.9.1 Design combination and value of actions

According to BS EN 1993-1-10 the design condition should consider the following combination of actions

$$A[T_{Ed}] + \sum G_k + \psi_1 Q_{k1} + \psi_{2,i} Q_{ki}$$

BS EN 1993-1-10 (2.1)

in which T_{Ed} is the reference temperature. For buildings the value of T_{Ed} for internal steelwork is given by the UK National Annex to BS EN 1993-1-1 as –5°C.

Here, for the above combination of actions, the design tension force is:

N_{Ed} = 95 kN

Dimensions of weld

Attachment 'length of weld' 6 mm (weld leg length)
Attachment 'width of weld' 100 mm (width of SHS)

Note: The weld dimensions are as defined in Table NA.1; 'length of weld' is measured in the direction of the tensile stress and 'width of weld' is measured transverse to the direction of the tensile stress.

Classify detail

The detail should be classified in terms of ΔT_{RD} following the guidance given in NA.2.1.1.2 of BS EN 1993-1-10.

PD 6695-1-10 2.2i)

The dimension of the welded attachment considered here fall outside of the limits given in Table NA.1 as the length is not applicable. Therefore,

BS EN 993-1-10 Table NA.1

ΔT_{RD} = 0°C

NA.2.1.1.2

For internal steelwork and ΔT_{RD} = 0°C the detail type is:
'Welded – moderate'

PD 6695-1-10 Table 2

Tensile stress level

The tensile stress in the SHS may be considered to be:

$$\sigma_{\mathrm{Ed}} = \frac{N_{\mathrm{Ed}}}{2b_{\mathrm{eff}}} = \frac{95\times10^{3}}{2\times68.5\times6.3} = 110\ \mathrm{N/mm^2}$$

The tensile stress level at the detail is:

PD 6695-1-10 2.2ii)

$$\frac{\sigma_{\mathrm{Ed}}}{f_{\mathrm{y}}(t)} = \frac{110}{355} = 0.31$$

Initial column in table

PD 6695-1-10 Table 2

For a ‘welded – moderate’ detail, the stress level (0.31) is between that for comb 6 and that for comb 7. Noting that 2.2vi) of PD 6695-1-10 allows interpolation between adjacent columns for ‘borderline cases’, take the initial column as comb 6 and interpolate to the right once the final column has been decided.

Adjustment to table column selection

Verify whether the initial table column selection needs to be altered for the criteria given in Note A to Table 2.

Charpy test temperature

NA.2.1.1.4 of the UK National Annex to BS EN 1993-1-10 give adjustments to the reference temperature based on the difference between the Charpy test temperature and the minimum steel temperature. These adjustments have been accounted for in the Tables given in PD 6695-1-10. Thus no alteration is required.

Gross stress concentration factor (ΔT_{Rg})

It is considered that there will be no gross stress concentration as the tensile stress level has been determined using an effective width acting on only two sides of the SHS. Therefore the criterion is met, thus

$\Delta T_{\mathrm{Rg}} = 0$

Radiation loss (ΔT_{r})

There is no radiation loss for the joint considered here. Therefore the criterion is met, thus

$\Delta T_{\mathrm{r}} = 0$

Strain rate ($\Delta T_{\dot{\varepsilon}}$)

Here the strain rate is not greater than to the reference strain rate given in BS EN 1993-1-5 ($\dot{\varepsilon} = 4\times10^{-4}$/sec Therefore the criterion is met, thus

$\Delta T_{\dot{\varepsilon}} = 0$

Cold forming ($\Delta T_{\varepsilon\mathrm{cf}}$)

The section considered here is hot finished, therefore no cold forming is present and the criterion is met, thus

$\Delta T_{\varepsilon\mathrm{cf}} = 0$

As all four criteria are met, the table column selection does not need to be adjusted.

Therefore for S355, 'welded – moderate' and interpolating between comb 6 $\left(\frac{\sigma_{Ed}}{f_y(t)} = 0.3\right)$ and comb 7 $\left(\frac{\sigma_{Ed}}{f_y(t)} \geq 0.5\right)$, the limiting steel thickness are:

PD 6695-1-10 Table 2

J0 54.5 mm
J2 81.8 mm

6.3mm < 54.5 mm < 81.8 mm

Therefore, an appropriate steel grade for the SHS is S355J0.

1.10 Blue Book approach

Page references in Section 1.10 are to P363 unless otherwise stated.

The bolt and welding resistances calculated in Sections 1.7 and 1.8 of this example could have been obtained from SCI publication P363. However, P363 does not contain values for the tension resistance of the cross section. Therefore the verifications given in Section 1.6 of this example still need to be carried out.

1.10.1 Design value of axial tension

$N_{Ed} = 140$ kN

1.10.2 Resistance of the bolts

Bearing resistance

The design bearing resistance of a single M20 non-preloaded class 8.8 bolt in S355 steel 8 mm thick ply, with $e_1 = 40$ mm and $e_2 = 30$ mm is:

$F_{b,Rd} = 77.2$ kN

Page D-303

Shear resistance

The design shear resistance of a single M20 non-preloaded class 8.8 bolt with a single shear plane is:

Page D-303

$F_{v,Rd} = 94.1$ kN

Resistance of a group of bolts

$F_{b,Rd} < F_{v,Rd}$

Therefore, the design resistance of the joint is:

$F_{Rd,joint} = 2F_{b,Rd} = 2 \times 77.2 = 154.4$ kN

$$\frac{N_{Ed}}{N_{Rd,joint}} = \frac{140}{154.4} = 0.91 < 1.0$$

Therefore the resistance of the bolts is adequate.

1.10.3 Resistance of the weld

For a fillet weld with a throat thickness of a = 4.2 mm (leg length of 6 mm). The design transverse resistance of the fillet weld is:

$F_{w,T,Rd}$ = 1.24 kN/mm — Page D-316

Note: This resistance value is greater than that determined in Section 1.8 of this example as the Blue Book uses the directional method to determine the transverse resistance of the weld compared with the simplified method used in Section 1.8.

The weld length l = 137 mm — Sheet 8

Therefore, the design weld force is:

$$F_{w,Ed} = \frac{N_{Ed}}{l} = \frac{140}{137} = 1.02 \text{ kN/mm}$$

$$\frac{F_{w,Ed}}{F_{w,t,Rd}} = \frac{1.02}{1.24} = 0.82 < 1.0$$

Therefore, the resistance of the fillet weld is adequate.

References are to BS EN 1993-1-1: 2005, including its National Annex, unless otherwise stated.

2 Pin-ended column

2.1 Scope

The pin-ended column shown in Figure 2.1 is subject to compression. Verify the adequacy of a hot finished 200 × 200 × 6.3 SHS in S355 steel.

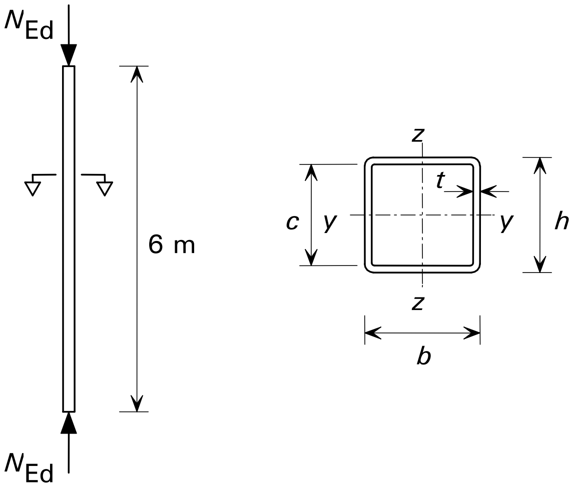


Figure 2.1

The design aspects covered in this example are:

- Cross section classification
- Cross-sectional resistance to axial compression
- Flexural buckling resistance.

2.2 Design force for ultimate limit state

Design compression force N_{Ed} = 920 kN

2.3 Section properties

Hot finished 200 × 200 × 6.3 SHS in S355 steel — P363

Depth of section	h	= 200 mm
Width of section	b	= 200 mm
Wall thickness	t	= 6.3 mm
Radius of gyration	i	= 7.89 cm
Cross sectional area	A	= 48.40 cm^2

For buildings that will be built in the UK the nominal values of the yield strength (f_y) and the ultimate strength (f_u) for structural steel should be those obtained from the product standard. Where a range is given the lowest nominal value should be used. NA.2.4

For S355 steel and $t \leq 16$ mm
Yield strength $f_y = R_{eH} = 355$ N/mm^2 BS EN 10210-1 Table A.3

2.4 Cross section classification

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81$$

Table 5.2

Internal compression parts

$$c = h - 3t = 200 - 3 \times 6.3 = 181.1 \text{ mm}$$

$$\frac{c}{t} = \frac{181.1}{6.3} = 28.75$$

Table 5.2

The limiting value for Class 1 is $\frac{c}{t} \leq 33\varepsilon = 33 \times 0.81 = 26.73$

The limiting value for Class 2 is $\frac{c}{t} \leq 38\varepsilon = 38 \times 0.81 = 30.78$

$26.73 < 28.75 < 30.78$ therefore the internal compression parts are Class 2.

As $b = h$ only one check is required; therefore the cross section is Class 2.

2.5 Partial factors for resistance

NA.2.15

$\gamma_{M0} = 1.0$

$\gamma_{M1} = 1.0$

2.6 Cross-sectional resistance

2.6.1 Compression resistance

Verify that 6.2.4(1) Eq (6.9)

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0$$

The design resistance of the cross section to compression is:

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}} \text{ (For Class 1, 2 and 3 cross sections)}$$

6.2.4(2) Eq (6.10)

$$N_{c,Rd} = \frac{4840 \times 355}{1.0} \times 10^{-3} = 1718 \text{ kN}$$

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{c,Rd}}} = \frac{920}{1718} = 0.54 < 1.0$$

6.2.4(1)
Eq (6.9)

Therefore the compressive resistance of the SHS cross section is adequate.

2.7 Flexural buckling resistance

Verify that:

6.3.1.1(1)
Eq (6.46)

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{b,Rd}}} \leq 1.0$$

The design buckling resistance is determined from:

6.3.1.1(3)
Eq (6.47)

$$N_{\mathrm{b,Rd}} = \frac{\chi A f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}} \text{(For Class 1 and 2 cross sections)}$$

χ is the reduction factor for the relevant buckling mode and is determined from:

6.3.1.2(1)

$$\chi = \frac{1}{(\Phi + \sqrt{(\Phi^2 - \overline{\lambda}^2})} \quad \text{but } \chi \leq 1.0$$

Eq (6.49)

Where:

$$\Phi = 0.5\left(1 + \alpha\left(\overline{\lambda} - 0.2\right) + \overline{\lambda}^2\right)$$

For hot finished SHS in S355 steel, use buckling curve '*a*'

Table 6.2
Table 6.1

For buckling curve '*a*' the imperfection factor $\alpha = 0.21$

For flexural buckling the slenderness is determined from:

6.3.1.3(1)
Eq (6.50)

$$\overline{\lambda} = \sqrt{\frac{Af_{\mathrm{y}}}{N_{\mathrm{cr}}}} = \left(\frac{L_{\mathrm{cr}}}{i}\right)\left(\frac{1}{\lambda_1}\right) \text{ (For Class 1, 2 and 3 cross sections)}$$

$$\lambda_1 = 93.9\varepsilon = 93.9 \times 0.81 = 76.1$$

The buckling length about both axes is $L_{\mathrm{cr}} = L = 6000$ mm

As the cross section is square $\overline{\lambda}_y = \overline{\lambda}_z$

$$\overline{\lambda} = \left(\frac{L_{\mathrm{cr}}}{i}\right)\left(\frac{1}{\lambda_1}\right) = \left(\frac{6000}{78.9}\right)\left(\frac{1}{76.1}\right) = 1.00$$

Eq (6.50)

6.3.1.2(1)

$$\Phi = 0.5\left(1 + \alpha\left(\overline{\lambda} - 0.2\right) + \overline{\lambda}^2\right) = 0.5 \times \left(1 + 0.21 \times \left(1.0 - 0.2\right) + 1.0^2\right) = 1.08$$

$$\chi = \frac{1}{(\Phi + \sqrt{(\Phi^2 - \overline{\lambda}^2})} = \frac{1}{1.08 + \sqrt{(1.08^2 - 1.0^2)}} = 0.67$$

Eq (6.49)

$0.67 < 1.0$

Therefore, $\chi = 0.67$

$N_{\mathrm{b,Rd}} = \dfrac{\chi A f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}} = \dfrac{0.67 \times 4840 \times 355}{1.0} \times 10^{-3} = 1151\ \mathrm{kN}$ — Eq (6.47)

$\dfrac{N_{\mathrm{Ed}}}{N_{\mathrm{b,Rd}}} = \dfrac{920}{1151} = 0.80 < 1.0$

Therefore, the flexural buckling resistance of SHS is adequate.

2.8 Blue Book Approach

Page references in Section 2.8 are to P363 unless otherwise stated.

The resistances calculated in Sections 2.6 and 2.7 of this example could have been obtained from SCI publication P363.

2.8.1 Design value of compression force

$N_{\mathrm{Ed}} = 920$ kN

2.8.2 Cross section classification

Under compression the cross section is at least Class 3. — Section 6.2(a) & Page D-17

2.8.3 Cross-sectional resistance

Resistance to compression

$N_{\mathrm{c,Rd}} = N_{\mathrm{pl,Rd}} = 1720$ kN — Page D-188

$\dfrac{N_{\mathrm{Ed}}}{N_{\mathrm{c,Rd}}} = \dfrac{920}{1720} = 0.53 < 1.0$

Therefore the resistance to compression is adequate.

2.8.4 Buckling resistance

Flexural buckling

The buckling length about both axes is $L_{\mathrm{cr}} = L = 6000$ mm

For buckling about both axes with a buckling length of 6.0 m, the buckling resistance is: — Page D-17

$N_{\mathrm{b,Rd}} = 1150$ kN

$\dfrac{N_{\mathrm{Ed}}}{N_{\mathrm{b,Rd}}} = \dfrac{920}{1150} = 0.80 < 1.0$

Therefore the flexural buckling resistance is adequate.

References are to BS EN 1993-1-1: 2005, including its National Annex, unless otherwise stated.

3 Simply supported laterally restrained beam

3.1 Scope

The beam shown in Figure 3.1 is fully restrained laterally along its length. Verify the adequacy of a hot finished 250 × 150 × 16 RHS in S355 steel for this beam.

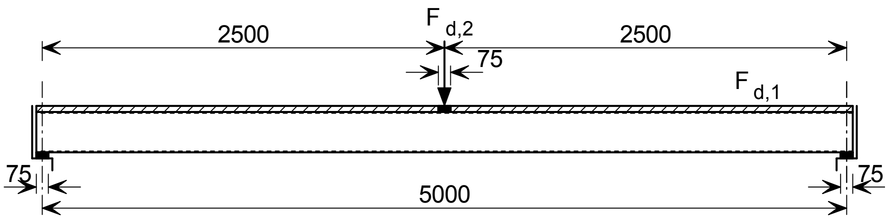


Figure 3.1

The design aspects covered in this example are:

- Calculation of design values of actions for ULS and SLS
- Cross section classification
- Cross-sectional resistance:
  - Shear buckling
  - Shear
  - Bending moment
- Resistance of web to transverse forces
- Vertical deflection of beam at SLS.

3.2 Actions (loading)

3.2.1 Permanent actions

Uniformly distributed load $g_1 = 3$ kN/m

Concentrated load $G_2 = 40$ kN

3.2.2 Variable actions

Uniformly distributed load $q_1 = 3$ kN/m

Concentrated load $Q_2 = 50$ kN

The variable actions given above are not due to storage and are not independent of each other.

There are no other variable actions to be considered.

3.2.3 Partial factors for actions

For the design of structural members not involving geotechnical actions, the partial factors for actions to be used for ultimate limit state strength verifications should be obtained from Table A1.2(B). Note 2 to Table A1.2(B) allows the National Annex to specify different values for the partial factors.

BS EN 1990 A1.3.1(4)

Partial factor for permanent actions $\gamma_G = 1.35$

Partial factor for variable actions $\gamma_Q = 1.50$

Reduction factor $\xi = 0.925$

BS EN 1990 Table NA.A1.2(B)

For this example the factor for the combination value of a variable action is: $\psi_0 = 0.7$

BS EN 1990 Table NA.A1.1

3.2.4 Design values of combined actions for ultimate limit state

BS EN 1990 presents two methods for determining the effects due to combination of actions for the ultimate limit state verification for the resistance of a structural member. The methods are to use expression (6.10) on its own or to determine the less favourable of the values from expressions (6.10a) and (6.10b).

Note 1 to Table NA.A1.2(B) in the UK National Annex to BS EN 1990 allows either method to be used.

Note: The two methods are briefly discussed in the introductory text to this publication.

The second method using expressions (6.10a) and (6.10b) is used here. Therefore the design values are taken as the most onerous values obtained from the following expressions:

BS EN 1990 Table A1.2(B)

$$\gamma_{\mathrm{Gj,sup}} G_{\mathrm{j,sup}} + \gamma_{\mathrm{Q,1}} \psi_{0,1} Q_1 + \gamma_{\mathrm{Q,i}} \psi_{0,i} Q_i \qquad (6.10a)$$

$$\xi \gamma_{\mathrm{Gj,sup}} G_{\mathrm{j,sup}} + \gamma_{\mathrm{Q,1}} Q_1 + \gamma_{\mathrm{Q,i}} \psi_{0,i} Q_i \qquad (6.10b)$$

Here Q_i is not required as the variable actions are not independent of each other and expression 6.10b gives the more onerous value. The design values are:

Combination of uniformly distributed loads

BS EN 1990 Table NA.A1.2(B) & Eq (6.10b)

$$F_{\mathrm{d,1}} = \xi \gamma_G g_1 + \gamma_Q q_1 = (0.925 \times 1.35 \times 3) + (1.5 \times 3) = 8.2 \text{ kN/m}$$

Combination of concentrated loads

$$F_{\mathrm{d,2}} = \xi \gamma_G G_2 + \gamma_Q Q_2 = (0.925 \times 1.35 \times 40) + (1.5 \times 50) = 125.0 \text{ kN}$$

3.3 Design bending moments and shear forces at ultimate limit state

Span of beam $L = 5000$ mm

Maximum value of the design bending moment occurs at the mid-span:

$$M_{\mathrm{Ed}} = \frac{F_{\mathrm{d,1}} L^2}{8} + \frac{F_{\mathrm{d,2}} L}{4} = \frac{8.2 \times 5^2}{8} + \frac{125 \times 5}{4} = 182.0 \text{ kNm}$$

Maximum design value of shear occurs at the supports:

$$V_{\mathrm{Ed}} = \frac{F_{\mathrm{d,1}} L}{2} + \frac{F_{\mathrm{d,2}}}{2} = \frac{8.2 \times 5}{2} + \frac{125}{2} = 83.0 \text{ kN}$$

Design value of shear force at the mid-span:

$$V_{\mathrm{Ed,C}} = V_{\mathrm{Ed}} - \frac{F_{\mathrm{d,1}} L}{2} = 83 - \frac{8.2 \times 5}{2} = 62.5 \text{ kN}$$

The design shear force and bending moment diagrams are shown in Figure 3.2.

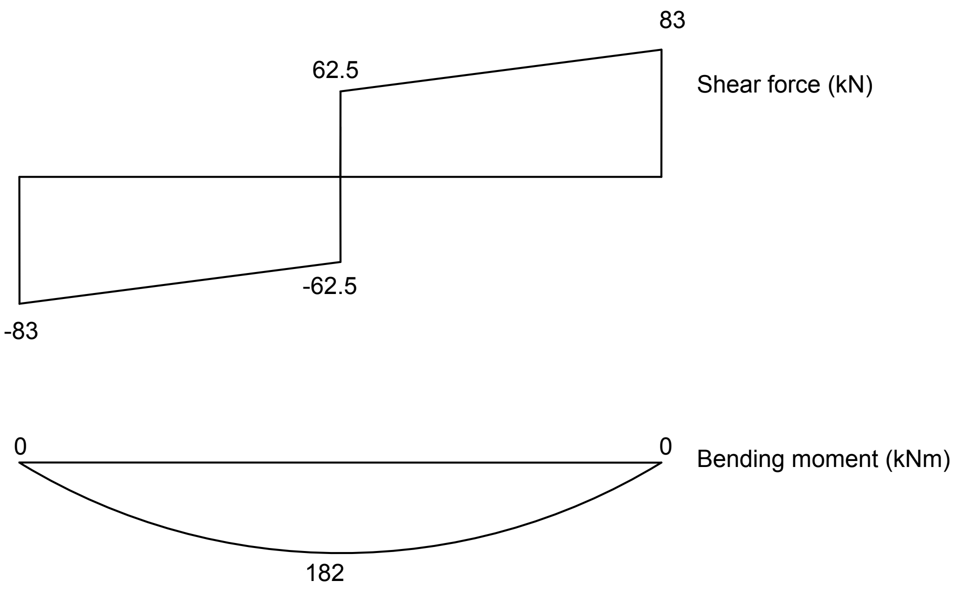


Figure 3.2

3.4 Section properties

For a hot finished 250 × 150 × 16 RHS in S355 steel: — P363

Depth of section	h	= 250.0 mm
Width of section	b	= 150.0 mm
Wall thickness	t	= 16.0 mm
Second moment of area about the y-axis	I_y	= 8880 cm^4
Radius of gyration about the y-axis	i_y	= 8.79 cm
Radius of gyration about the z-axis	i_z	= 5.80 cm
Plastic modulus about the y-axis	$W_{\mathrm{pl,y}}$	= 906 cm^3
Cross-sectional area	A	= 115 cm^2

For buildings that will be built in the UK the nominal values of the yield strength (f_y) and the ultimate strength (f_u) for structural steel should be those obtained from the product standard. Where a range is given the lowest nominal value should be used. — NA.2.4

For S355 steel and $t \leq 16$ mm: — BS EN 10210-1 Table A.3

Yield strength $f_y = R_{\mathrm{eH}} = 355$ N/mm^2

3.5 Cross section classification

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81$$ Table 5.2

Internal part subject to bending (web)

$$c = h - 3t = 250 - 3 \times 16 = 202 \text{ mm}$$

$$\frac{c}{t} = \frac{202}{16} = 12.63$$

The limiting value for Class 1 is $\frac{c}{t} \leq 72\varepsilon = 72 \times 0.81 = 58.32$ Table 5.2

12.63 < 58.32 therefore the internal part in bending is Class 1.

Internal part subject to compression (flange) Table 5.2

$$c = b - 3t = 150 - 3 \times 16 = 102 \text{ mm}$$

$$\frac{c}{t} = \frac{102}{16} = 6.38$$

The limiting value for Class 1 is $\frac{c}{t} \leq 33\varepsilon = 33 \times 0.81 = 26.73$

6.38 < 26.73 therefore the internal part in compression is Class 1.

Therefore the section is Class 1 for bending about the y-y axis.

3.6 Partial factors for resistance

$\gamma_{M0} = 1.0$ NA.2.15

$\gamma_{M1} = 1.0$

3.7 Cross-sectional resistance

3.7.1 Shear buckling

The shear buckling resistance for webs should be verified according to Section 5 of BS EN 1993-1-5 if: 6.2.6(6)

$$\frac{h_w}{t_w} > \frac{72\varepsilon}{\eta}$$ Eq (6.22)

$\eta = 1.0$ (conservative)

$h_w = h - 2t = 250 - 2 \times 16 = 218$ mm

$$\frac{h_w}{t_w} = \frac{218}{16} = 13.6$$

$$\frac{72\varepsilon}{\eta} = \frac{72 \times 0.81}{1.0} = 58.3$$

13.6 < 58.3

Therefore the shear buckling resistance of the web does not need to be verified.

3.7.2 Shear resistance

Verify that: (6.2.6(1) Eq (6.17))

$$\frac{V_{Ed}}{V_{c,Rd}} \leq 1.0$$

For plastic design $V_{c,Rd}$ is the design plastic shear resistance ($V_{pl,Rd}$).

$$V_{c,Rd} = V_{pl,Rd} = \frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}}$$

6.2.6(2) Eq (6.18)

A_v is the shear area and is determined as follows for rolled RHS sections with the load applied parallel to the depth.

$$A_v = \frac{Ah}{b+h} = \frac{11500 \times 250}{150 + 250} = 7187.5 \text{ mm}^2$$

6.2.6(3)(f)

$$V_{c,Rd} = \frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}} = \frac{7187.5 \times (355/\sqrt{3})}{1.0} \times 10^{-3} = 1473 \text{ kN}$$

Eq (6.18)

Maximum design shear $V_{Ed} = 83.0$ kN (Sheet 3)

$$\frac{V_{Ed}}{V_{c,Rd}} = \frac{83}{1473} = 0.06 < 1.0$$

Therefore the shear resistance of the RHS is adequate.

3.7.3 Resistance to bending

Verify that:

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0$$

6.2.5(1) Eq (6.12)

At the point of maximum bending moment (mid-span) check whether the shear force will reduce the bending resistance of the section. (6.2.8(2))

$$\frac{V_{c,Rd}}{2} = \frac{1473}{2} = 736.5 \text{ kN}$$

$$V_{Ed,C} = 62.5 \text{ kN} < 736.5 \text{ kN}$$

Therefore **no reduction** in bending resistance due to shear is required.

The design resistance for bending for Class 1 and 2 cross sections is: 6.2.5(2)

$$M_{\mathrm{c,Rd}} = M_{\mathrm{pl,Rd}} = \frac{W_{\mathrm{pl,y}} f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} = \frac{906 \times 10^3 \times 355}{1.0} \times 10^{-6} = 322 \text{ kNm}$$

Eq (6.13)

$$\frac{M_{\mathrm{Ed}}}{M_{\mathrm{c,Rd}}} = \frac{182}{322} = 0.57 < 1.0$$

6.2.5(1)
Eq (6.12)

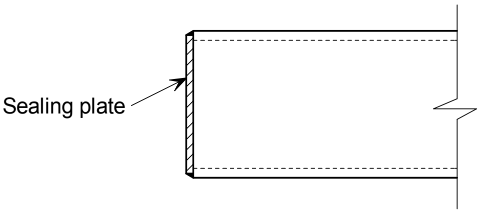
Therefore the bending resistance is adequate.

3.8 Resistance of the web to transverse forces

P363

The design verification given in BS EN 1993-1-5 does not relate to closed hollow sections. Therefore, a method based on established practice is used.

The design resistance of the web to transverse forces (F_{Rd}) should be taken as the smaller of the bearing ($F_{\mathrm{Rd,bearing}}$) and buckling ($F_{\mathrm{Rd,buckling}}$) resistances of the web.



End detail

Figure 3.3

As there are sealing plates welded to the ends of the RHS the bearing resistance of the webs may be determined from.

$$F_{\mathrm{Rd,bearing}} = \left(b_1 + nk\right)\frac{2t f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}$$

P363 9.2(a)

where:

b_1 is the stiff bearing length

$b_1 = 75$ mm

$n = 2$ for end bearing or $n = 5$ for continuous over bearing

Therefore,

$n = 2$

$k = t$ for hollow sections, thus $k = 16$ mm

$f_{\mathrm{y}} = 355$ N/mm^2 Sheet 3

$$F_{\mathrm{Rd,bearing}} = \left(75 + (2 \times 16)\right) \times 2 \times 16 \times \frac{355}{1.0} \times 10^{-3} = 1216 \text{ kN}$$

As flange plates are welded to the RHS , the buckling resistance ($F_{\mathrm{Rd,buckling}}$) of the two webs is determined as follows:

$$F_{\mathrm{Rd,buckling}} = \left(b_1 + n_1\right) 2t\chi \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}$$

Derived from
P363 9.2(b)

where:

$b_1 = 75$ mm — P363 9.2(b)

n_1 = is the length obtained by dispersion at 45° through half depth of the section

$$n_1 = \frac{h}{2} = \frac{250}{2} = 125 \text{ mm}$$

t = 16 mm

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \le 1.0$$

6.3.1.2(1)

$$\Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right]$$

$$\bar{\lambda} = \sqrt{\frac{\lambda^2 f_y}{\pi^2 E}}$$

$$\lambda = 1.5\left(\frac{h-2t}{t}\right)\sqrt{3} = 1.5 \times \left(\frac{250-(2\times16)}{16}\right) \times \sqrt{3} = 35.4$$

$$\bar{\lambda} = \sqrt{\frac{35.4^2 \times 355}{\pi^2 \times 210000}} = 0.46$$

Each web may be considered as a solid rectangular section. Therefore use buckling curve 'c' — Table 6.2

For buckling curve 'c' $\alpha = 0.49$ — Table 6.1

6.3.1.2(1)

$$\Phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right] = 0.5 \times \left[1 + 0.49 \times (0.46 - 0.2) + 0.46^2\right] = 0.67$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = \frac{1}{0.67 + \sqrt{0.67^2 - 0.46^2}} = 0.86 < 1.0$$

Therefore

$$\chi = 0.86$$

$$F_{\mathrm{Rd,buckling}} = (b_1 + n_1)2t\chi\frac{f_y}{\gamma_{\mathrm{M0}}}$$

$$= (75 + 125) \times 2 \times 16 \times 0.86 \times \frac{135}{1.0} \times 10^{-3} = 1954 \text{ kN}$$

$$F_{\mathrm{Rd,bearing}} = 1216 \text{ kN} < F_{\mathrm{Rd,buckling}} = 1954 \text{ kN}$$

Therefore, the resistance of the two webs of the RHS to transverse forces is:

$$F_{\mathrm{Rd}} = 1216 \text{ kN}$$

$$\frac{V_{\mathrm{Ed}}}{F_{\mathrm{Rd}}} = \frac{83}{1216} = 0.07 < 1.0$$

The resistance of the web to transverse forces is adequate.

3.9 Vertical deflection at serviceability limit state

A structure should be designed and constructed such that all relevant serviceability criteria are satisfied. 7.1(1)

No specific requirements at SLS are given in BS EN 1993-1-1, 7.1; it is left for the project to specify the limits, associated actions and analysis model. Guidance on the selection of criteria is given in BS EN 1990, A.1.4.

For this example, the only serviceability limit state that is to be considered is the vertical deflection under variable actions, because excessive deflection would damage brittle finishes that are added after the permanent actions have occurred. The limiting deflection for this beam is taken to be span/360, which is consistent with common design practice.

3.9.1 Design values of actions

As noted in BS EN 1990, the SLS partial factors on actions are taken as unity and expression 6.14a is used to determine design effects. Additionally, as stated in Section 3.2.2, the variable actions are not independent and therefore no combination factors (ψ_i) are required. Thus the combination values of actions are given by: BS EN 1990 A1.4.1(1)

$F_{\mathrm{d,1,ser}} = g_1 + q_1$ and $F_{\mathrm{d,2,ser}} = G_2 + Q_2$

As noted above, the permanent actions considered in this example occur during the construction process, therefore only the variable actions need to be considered in the serviceability verification for the functioning of the structure. BS EN 1990 A1.4.3(3)

Thus $F_{\mathrm{d,1,ser}} = q_1 = 3.0$ kN/m and $F_{\mathrm{d,2,ser}} = Q_2 = 50.0$ kN

Therefore the vertical deflection is given by:

$$w = \left(\frac{1}{EI_{\mathrm{y}}}\right)\left(\frac{5F_{\mathrm{d,1,}ser}L^4}{384} + \frac{F_{\mathrm{d,2,ser}}L^3}{48}\right)$$

Modulus of elasticity $E = 210000$ N/mm^2 3.2.6(1)

$$w = \left(\frac{1}{210000 \times 8880 \times 10^4}\right)\left(\frac{5 \times 3 \times 5000^4}{384} + \frac{50 \times 10^3 \times 5000^3}{48}\right) = 8.3 \text{ mm}$$

The vertical deflection limit is:

$$w_{\mathrm{lim}} = \frac{L}{360} = \frac{5000}{360} = 13.9 \text{ mm}$$

8.3 mm < 13.9 mm

Therefore the vertical deflection of the beam is satisfactory.

3.10 Blue Book Approach

Page references given in Section 3.10 are to P363 unless otherwise stated.

The resistances calculated in Sections 3.7.2, 3.7.3 and 3.8 of this example could have been obtained from SCI publication P363.

3.10.1 Design moments and shear forces at ultimate limit state

Maximum design bending moment occurs at the mid-span:

$M_{\mathrm{Ed}} = 182.0$ kNm Sheet 3

Maximum design shear force occurs at the supports:

$V_{Ed} = 83.0$ kN — Sheet 3

Design shear force at the mid-span:

$V_{Ed,C} = 62.5$ kN — Sheet 3

3.10.2 Cross section classification

Under bending about the major axis (*y-y*) the cross section is Class 1. — Page D-87

3.10.3 Cross-sectional resistance

Shear resistance

$V_{c,Rd} = 1470$ kN — Page D-87

$$\frac{V_{Ed}}{V_{c,Rd}} = \frac{83}{1470} = 0.06 < 1.0$$

Therefore the shear resistance is adequate.

Bending resistance

$$\frac{V_{c,Rd}}{2} = \frac{1470}{2} = 735 \text{ kN}$$

$V_{Ed,C} = 62.5$ kN < 735 kN

Therefore the shear is low and $M_{c,y,Rd} = 322$ kNm — Page D-87

$$\frac{M_{Ed}}{M_{c,y,Rd}} = \frac{182}{322} = 0.57 < 1.0$$

Therefore the bending resistance is adequate>

3.10.4 Resistance of the web to transverse forces at the end of the beam

The Blue Book gives separate verifications for the bearing and buckling resistance of the webs.

Bearing resistance

$F_{Rd,bearing} = b_1 C_2 + C_1$ — Section 9.2(a)

$b_1 = 75$ mm — Sheet 6

For end bearing: — Page D-117

$C_1 = 364$

$C_2 = 11.4$

$F_{Rd,bearing} = (75 \times 11.4) + 364 = 1219$ kN

Buckling resistance

$F_{\mathrm{Rd,buckling}} = b_1 C_2 + C_1$

Section 9.2(b)
Page D-117

Without welded flange plates to the bottom of the rolled hollow section:

$C_1 = 1200$

$C_2 = 4.82$

$F_{\mathrm{Rd,buckling}} = (75 \times 4.82) + 1200 = 1561 \text{ kN}$

$F_{\mathrm{Rd,bearing}} = 1219 \text{ kN} < F_{\mathrm{Rd,buckling}}$ 1561 kN

Therefore the bearing resistance is critical.

$$\frac{F_{\mathrm{Ed}}}{F_{\mathrm{Rd,bearing}}} = \frac{83}{1219} = 0.07 < 1.0$$

Note

The Blue Book (P363) does not include deflection values, so the SLS deflection verification must be carried out as in Section 3.9 of this example.

4 Laterally unrestrained beam

References are to BS EN 1993-1-1: 2005, including its National Annex, unless otherwise stated.

4.1 Scope

Consider the beam given in Example 3 but without lateral restraint, other than at the supports.

In addition to all the verifications for a restrained beam given in Example 3, a laterally unrestrained beam must be checked for lateral torsional buckling.

Lateral torsional buckling is usually the mode of failure for laterally unrestrained open section beams. Circular and square hollow sections are not usually susceptible to this failure mode, but, in unusual situations only, rectangular hollow sections which have a large slenderness ($\bar{\lambda}_{LT}$) are susceptible to lateral torsional buckling.

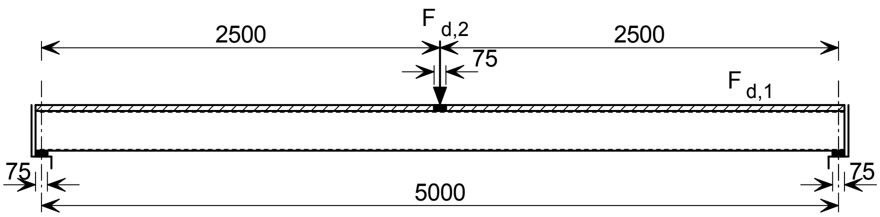


Figure 4.1

The design aspect covered in this example is:

- Lateral torsional buckling

4.2 Design shear force and moments at ultimate limit state

The design shear force and bending moment diagrams at ULS are the same as in Example 3 and are shown in Figure 4.2.

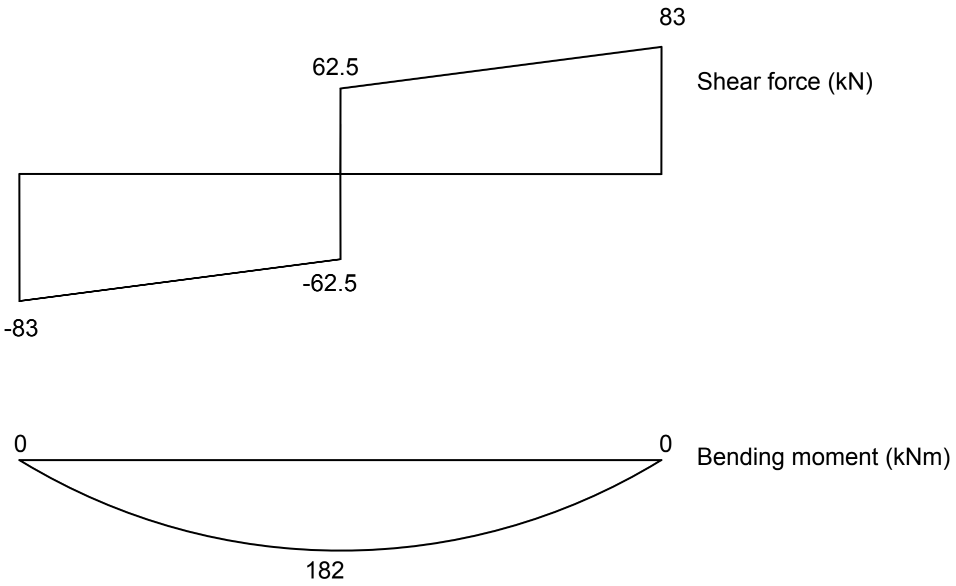


Figure 4.2

4.3 Section properties

For a hot finished 250 × 150 × 16 RHS in S355 steel (the same section as in Example 3): P363

Depth of section	h	= 250.0 mm
Width of section	b	= 150.0 mm
Wall thickness	t	= 16.0 mm
Second moment of area about the y-axis	I_y	= 8880 cm^4
Second moment of area about the z-axis	I_z	= 3870 cm^4
Plastic modulus about the y-axis	$W_{pl,y}$	= 906 cm^3
St Venant torsional constant	I_T	= 8870 cm^4

For buildings that will be built in the UK, the nominal values of the yield strength (f_y) and the ultimate strength (f_u) for structural steel should be those obtained from the product standard. Where a range is given, the lowest nominal value should be used. NA.2.4

For S355 steel and $t \leq 16$ mm: BS EN 10210-1 Table A.3

Yield strength $f_y = R_{eH} = 355$ N/mm^2

4.4 Material properties

Modulus of elasticity $E = 210000$ N/mm^2 — 3.2.6(1)

Shear modulus $G = 81000$ N/mm^2

4.5 Lateral torsional buckling

If the slenderness for lateral torsional buckling ($\bar{\lambda}_{\mathrm{LT}}$) is less than or equal to $\bar{\lambda}_{\mathrm{LT,0}}$ the effects of lateral torsional buckling may be neglected, and only cross-sectional verifications apply. — 6.3.2.2(4)

In the UK National Annex the value of $\bar{\lambda}_{\mathrm{LT,0}}$ for rolled sections is given as $\bar{\lambda}_{\mathrm{LT,0}} = 0.4$ — NA.2.17

$$\bar{\lambda}_{\mathrm{LT}} = \sqrt{\frac{W_{\mathrm{y}} \times f_{\mathrm{y}}}{M_{\mathrm{cr}}}}$$ — 6.3.2.2(1)

$W_{\mathrm{y}} = W_{\mathrm{pl,y}}$ For class 1 or 2 cross sections.

BS EN 1993-1-1 does not give an expression for the determination of the elastic critical buckling moment for lateral torsional buckling (M_{cr}).

Access Steel document SN003 presents expressions that may be used to determine M_{cr}.

The general expression for doubly symmetrical sections is: — Access Steel document SN003

$$M_{\mathrm{cr}} = C_1 \frac{\pi^2 EI_{\mathrm{z}}}{(kL)^2} \left\{ \sqrt{\left(\frac{k}{k_{\mathrm{w}}}\right)^2 \frac{I_{\mathrm{w}}}{I_{\mathrm{z}}} + \frac{(kL)^2 GI_{\mathrm{T}}}{\pi^2 EI_{\mathrm{z}}} + (C_2 z_{\mathrm{g}})^2} - C_2 z_{\mathrm{g}} \right\}$$

where:

- L is the member length between points of lateral restraint
- z_{g} is the distance between the shear centre and the point of application of the transverse loading
- k & k_{w} are effective length factors
- C_1 & C_2 are coefficients which depend on the shape of the bending moment and the end restraint conditions
- k_{w} = 1.0 (unless advised otherwise)
- k = 1.0 (for simply supported members).

For RHS the effects of warping $\left(\frac{I_{\mathrm{w}}}{I_{\mathrm{z}}}\right)$ are negligible compared with the effects of torsion $\left(\frac{(kL)^2 GI_{\mathrm{T}}}{\pi^2 EI_{\mathrm{z}}}\right)$, so may be neglected when determining M_{cr}.

Also, the effect of applying of the actions above or below the shear centre of the section may be neglected for RHS sections (i.e. take $z_{\mathrm{g}} = 0$).

Therefore, the terms $\left(\frac{k}{k_{\mathrm{w}}}\right)\frac{I_{\mathrm{w}}}{I_{\mathrm{z}}}$ and $C_2 z_{\mathrm{g}}$ may be removed from the expression and the following simplified expression used for RHS sections:

$$M_{cr} = C_1 \frac{\pi^2 EI_z}{(kL)^2} \sqrt{\frac{(kL)^2 GI_T}{\pi^2 EI_z}}$$

For the design bending moment shown in Figure 4.2, C_1 is:

Access Steel document SN003 Table 3.2

$C_1 = 1.127$

$$M_{cr} = 1.127 \times \left(\frac{\pi^2 \times 210000 \times 3870 \times 10^4}{(1 \times 5000)^2} \right)$$

$$\times \left(\sqrt{\frac{(1 \times 5000)^2 \times 81000 \times 8870 \times 10^4}{\pi^2 \times 210000 \times 3870 \times 10^4}} \right) = 5500 \times 10^6 \text{ Nmm}$$

6.3.2.2(1)

$$\bar{\lambda}_{LT} = \sqrt{\frac{W_y \times f_y}{M_{cr}}} = \sqrt{\frac{906 \times 10^3 \times 355}{5500 \times 10^6}} = 0.24$$

6.3.2.2(4)

Since $\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ (0.24 < 0.4) the lateral torsional buckling effects are neglected and only cross-sectional verifications apply.

Since the cross-sectional verifications are satisfied in Example 3, the 250 × 150 × 16 RHS in S355 steel is adequate to be unrestrained between the supports shown in Figure 4.1.

4.6 Blue Book Approach

Page references in Section 4.6 are to P363 unless otherwise stated.

The verification given in Section 4.5 of this example could have been obtained from SCI publication P363.

4.6.1 Design moment at ultimate limit state

Maximum design moment occurs at the mid-span

$M_{Ed} = 182$ kNm

Sheet 2

4.6.2 Cross section classification

Under bending about the major axis (*y-y*) the cross section is Class 1.

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4.6.3 Lateral torsional buckling resistance

The limiting length (L_c) above which the lateral torsional bucking resistance should be verified is:

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$L_c = 11.9$ m

$L = 5$ m < 11.9 m

Therefore the lateral torsional buckling effects may be neglected.

References are to BS EN 1993-1-1: 2005, including its National Annex, unless otherwise stated.

5 SHS subject to combined compression and bi-axial bending

5.1 Scope

Verify the adequacy of a hot finished SHS in S355 steel to resist the compression and bending about both axes shown in Figure 5.1.

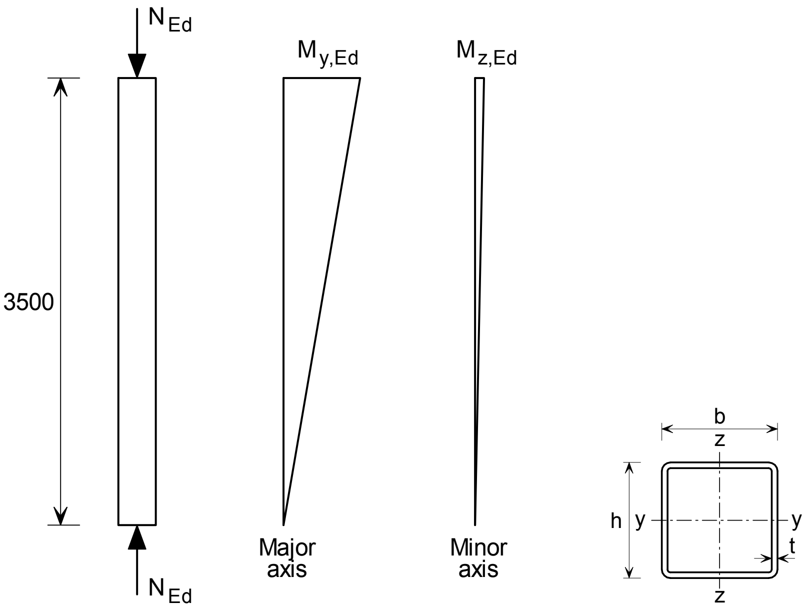

Figure 5.1

The design aspects covered in this example are:

- Cross section classification
- Cross-sectional resistance to combined compression and bi-axial bending
- Buckling resistance for combined compression and bi-axial bending using the verifications given in 6.3.3 of BS EN 1993-1-1.

5.2 Design force and moments at ultimate limit state

Design compression force	N_{Ed}	=	600 kN
Design Moment about the *y-y* axis (major axis)	$M_{\mathrm{y,Ed}}$	=	20 kNm
Design Moment about the *z-z* axis (minor axis)	$M_{\mathrm{z,Ed}}$	=	5 kNm

5.3 Section properties

For a hot finished 150 × 150 × 6.3 SHS in S355 steel: — P363

Depth of section	h	= 150 mm
Width of section	b	= 150 mm
Wall thickness	t	= 6.3 mm
Second moment of area	I	= 1223 cm^4
Radius of gyration	i	= 5.85 cm
Elastic modulus	W_{el}	= 163 cm^3
Plastic modulus	W_{pl}	= 192 cm^3
Area	A	= 35.8 cm^2
Modulus of elasticity	E	= 210000 N/mm^2 — 3.2.6(1)

For buildings that will be built in the UK, the nominal values of the yield strength (f_y) and the ultimate strength (f_u) for structural steel should be those obtained from the product standard. Where a range is given the lowest nominal value should be used. — NA.2.4

For S355 steel and $t \leq 16$ mm: — BS EN 10210-1 Table A.3

Yield strength $f_y = R_{eH} = 355\ N/mm^2$

5.4 Cross section classification

Table 5.2

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81$$

An elastic stress distribution in a SHS under combined compression and biaxial bending can be sketched as shown in Figure 5.2. The classification of the section may be conservatively determined by using the limits in the 'part subject to compression' section of Table 5.2 given in BS EN 1993-1-1.

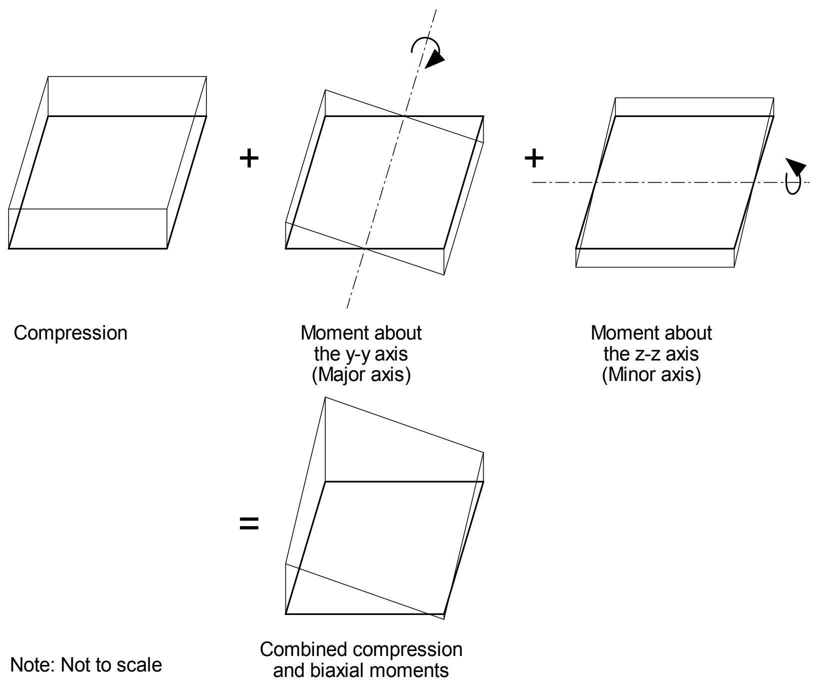

Figure 5.2

Internal compression part.

$c = h - 3t = 150 - 3 \times 6.3 = 131.1$ mm

$$\frac{c}{t} = \frac{131.1}{6.3} = 20.81$$

The most onerous limit is for Class 1, which is,

Table 5.2

$$\frac{c}{t} \leq 33\varepsilon = 33 \times 0.81 = 26.73$$

20.81 < 26.73

Therefore, the cross section is Class 1 under combined compression and bi-axial bending.

5.5 Partial factors for resistance

NA.2.15

$\gamma_{M0} = 1.0$

$\gamma_{M1} = 1.0$

5.6 Cross-sectional resistance

5.6.1 Bending and axial force

Verify that:

6.2.9.1(1) Eq (6.31)

$$M_{Ed} \leq M_{N,Rd}$$

For bi-axial bending, verify that:

6.2.9.1(6) Eq (6.41)

$$\left(\frac{M_{y,Ed}}{M_{N,y,Rd}}\right)^{\alpha} + \left(\frac{M_{z,Ed}}{M_{N,z,Rd}}\right)^{\beta} \leq 1.0$$

where:

$M_{N,Rd}$ is the design plastic moment resistance reduced due to the axial force.

For a square hollow section:

$$M_{N,Rd} = M_{N,y,Rd} = M_{N,z,Rd}$$

Therefore, where fastener holes are not to be accounted for:

6.2.9.1(5)

$$M_{N,Rd} = M_{N,y,Rd} \quad M_{pl,y,Rd} \frac{1-n}{1-0.5a_w} \quad \text{but} \quad M_{N,y,Rd} \leq M_{pl,y,Rd}$$

The design bending moment resistance of the cross section ($M_{pl,Rd}$) is:

6.2.5(2) (6.13)

$$M_{pl,y,Rd} = \frac{W_{pl,y} f_y}{\gamma_{M0}} = \frac{192 \times 10^3 \times 355}{1.0} \times 10^{-6} = 68 \text{ kNm}$$

6.2.9.1(5)

$$n = \frac{N_{Ed}}{N_{pl,Rd}}$$

The design resistance of the cross section to compression ($N_{pl,Rd}$) is:	6.2.3(2)
$N_{pl,Rd} = \dfrac{A f_y}{\gamma_{M0}} = \dfrac{3580 \times 355}{1.0} \times 10^{-3} = 1271$ kN	Eq (6.6)
$n = \dfrac{600}{1271} = 0.47$	6.2.9.1(5)
$a_w = \dfrac{A - 2bt}{A}$ but $a_w \leq 0.5$ (for hollow sections)	
$a_w = \dfrac{3580 - (2 \times 150 \times 6.3)}{3580} = 0.47 < 0.5$	
As $0.47 < 0.5$	
$a_w = 0.47$	
$M_{N,y,Rd} = M_{pl,y,Rd} \dfrac{1 - n}{1 - 0.5 a_w} = 68 \times \dfrac{1 - 0.47}{1 - (0.5 \times 0.47)} = 47$ kNm	6.2.9.1(5)
47 kNm < 68 kNm = $M_{pl,y,Rd}$	
Therefore, $M_{N,Rd} = M_{N,y,Rd} = 47$ kNm	
$M_{y,Ed} = 20$ kNm < 47 kNm	6.2.9.1(1)
$M_{z,Ed} = 5$ kNm < 47 kNm	Eq (6.31)
Therefore the cross-sectional resistance is satisfactory for each effect separately.	
Combined resistance to bi-axial bending and axial force:	
For SHS	6.2.9.1(6)
$\alpha = \beta = \dfrac{1.66}{1 - 1.13 n^2}$ but $\alpha = \beta \leq 6$	
$\alpha = \beta = \dfrac{1.66}{1 - (1.13 \times 0.47^2)} = 2.21$	
$2.21 < 6$	
Therefore, $\alpha = \beta = 2.21$	
$\left(\dfrac{M_{y,Ed}}{M_{N,y,Rd}}\right)^{\alpha} + \left(\dfrac{M_{z,Ed}}{M_{N,z,Rd}}\right)^{\beta} = \left(\dfrac{20}{47}\right)^{2.21} + \left(\dfrac{5}{47}\right)^{2.21} = 0.16 < 1.0$	6.2.9.1(6) Eq (6.41)
Therefore, the cross-sectional resistance to combined compression and bending is adequate.	

5.7 Buckling resistance of member

5.7.1 Combined bending and axial compression

The following criteria should be satisfied.

$$\frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{y}} N_{\mathrm{Rk}} / \gamma_{\mathrm{M1}}} + k_{\mathrm{yy}} \frac{M_{\mathrm{y.Ed}} + \Delta M_{\mathrm{y.Ed}}}{\chi_{\mathrm{LT}} M_{\mathrm{y.Rk}} / \gamma_{\mathrm{M1}}} + k_{\mathrm{yz}} \frac{M_{\mathrm{z.Ed}} + \Delta M_{\mathrm{z.Ed}}}{M_{\mathrm{z.Rk}} / \gamma_{\mathrm{M1}}} \le 1.0$$

Eq (6.61)

$$\frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{z}} N_{\mathrm{Rk}} / \gamma_{\mathrm{M1}}} + k_{\mathrm{zy}} \frac{M_{\mathrm{y.Ed}} + \Delta M_{\mathrm{y.Ed}}}{\chi_{\mathrm{LT}} M_{\mathrm{y.Rk}} / \gamma_{\mathrm{M1}}} + k_{\mathrm{zz}} \frac{M_{\mathrm{z.Ed}} + \Delta M_{\mathrm{z.Ed}}}{M_{\mathrm{z.Rk}} / \gamma_{\mathrm{M1}}} \le 1.0$$

Eq (6.62)

where:

For Class 1, 2 and 3 cross sections $\Delta M_{\mathrm{y,Ed}}$ and $\Delta M_{\mathrm{z,Ed}}$ are zero.

For Class 1 cross sections

$$N_{\mathrm{Rk}} = Af_{\mathrm{y}} = 3580 \times 355 \times 10^{-3} = 1271 \text{ kN}$$

Table 6.7

$$M_{\mathrm{z,Rk}} = M_{\mathrm{y,Rk}} = W_{\mathrm{pl,y}} f_{\mathrm{y}} = 192 \times 10^{3} \times 355 \times 10^{-6} = 68 \text{ kNm}$$

For square hollow sections with flexural buckling lengths that are equal about both axes:

$$\chi_{\mathrm{y}} = \chi_{\mathrm{z}}$$

For flexural buckling

6.3.1.2(1)
Eq (6.49)

$$\chi = \frac{1}{(\Phi + \sqrt{(\Phi^2 - \overline{\lambda}^2)}} \le 1.0$$

where:

$$\overline{\lambda} = \sqrt{\frac{Af_{\mathrm{y}}}{N_{\mathrm{cr}}}} = \left(\frac{L_{\mathrm{cr}}}{i}\right)\left(\frac{1}{\lambda_1}\right)$$ (For Class 1, 2 and 3 cross sections)

6.3.1.3(1)
Eq (6.50)

For this example it is assumed that the buckling length about both axes is:

$L_{\mathrm{cr}} = L = 3500$ mm

$$\lambda_1 = 93.9\varepsilon = 93.9 \times 0.81 = 76$$

$$\overline{\lambda} = \left(\frac{3500}{58.5}\right) \times \left(\frac{1}{76}\right) = 0.79$$

For a hot finished SHS in S355 steel, use buckling curve ‘a’ — Table 6.2

For curve ‘a’ the imperfection factor is $\alpha = 0.21$ — Table 6.1

$$\Phi = 0.5\left[1 + \alpha\left(\overline{\lambda} - 0.2\right) + \overline{\lambda}^2\right] = 0.5 \times \left[1 + 0.21 \times \left(0.79 - 0.2\right) + 0.79^2\right] = 0.87$$

6.3.1.2(1)

$$\chi = \frac{1}{(\Phi + \sqrt{(\Phi^2 - \overline{\lambda}^2)}} = \frac{1}{0.87 + \sqrt{(0.87^2 - 0.79^2)}} = 0.81$$

Eq (6.49)

$0.81 < 1.0$

Therefore, $\chi = 0.81$

Square hollow sections are not susceptible to failure by lateral torsional buckling. 6.3.2.1(2)

Therefore, the lateral torsional buckling reduction factor is:

$\chi_{LT} = 1.0$

For sections not susceptible to torsional deformation use Table B.1. to determine the interaction factors k_{yy}, k_{zz}, k_{yz} and k_{zy}.

For class 1 and 2 cross sections Table B.1

$$k_{yy} = C_{my}\left[1+\left(\bar{\lambda}-0.2\right)\left(\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right] \quad \text{but}$$

$$k_{yy} \le C_{my}\left[1+\left(\frac{0.8N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right]$$

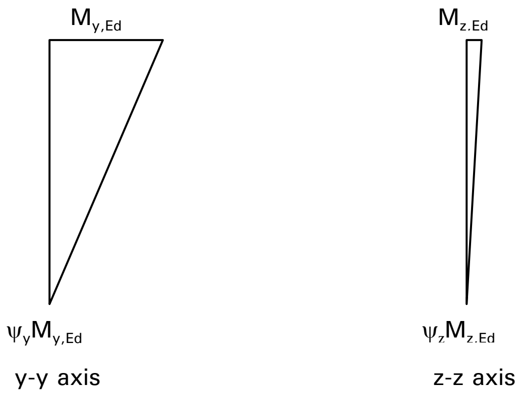

Figure 5.3

From the bending moment diagrams for both the *y-y* and *z-z* axes $\psi = 0$ Table B.3

Therefore,

$C_{my} = C_{mz} = C_{mLT} = 0.6 + 0.4\psi \ge 0.4$

$C_{my} = C_{mz} = C_{mLT} = 0.6 > 0.4$

Therefore,

$C_{my} = C_{mz} = C_{mLT} = 0.6$

Table B.1

$$k_{yy} = C_{my}\left[1+\left(\bar{\lambda}_y-0.2\right)\left(\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right]$$

$$= 0.6\times\left[1+\left(0.79-0.2\right)\times\left(\frac{600}{0.81\times 1271/1.0}\right)\right] = 0.81$$

but

$$k_{yy} \le C_{my}\left[1+\left(0.8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right]$$

$$= 0.6\times\left[1+\left(0.8\times\frac{600}{0.81\times1271/1}\right)\right] = 0.88$$

$0.81 < 0.88$

Therefore $k_{yy} = 0.81$

$k_{zy} = 0.6 \times k_{yy} = 0.6 \times 0.81 = 0.49$

By inspection of the formula given in Table B.1 it can be seen that for a SHS

$k_{zz} = k_{yy} = 0.81$

$k_{yz} = k_{zy} = 0.49$

$$\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}} + k_{yy}\frac{M_{y.Ed}+\Delta M_{y.Ed}}{\chi_{LT}M_{y.Rk}/\gamma_{M1}} + k_{yz}\frac{M_{z.Ed}+\Delta M_{z.Ed}}{M_{z.Rk}/\gamma_{M1}} \le 1.0$$

Eq (6.61)

$$\left(\frac{600}{0.81\times1271/1.0}\right)+0.81\times\left(\frac{20+0}{1.0\times68/1.0}\right)+0.49\times\left(\frac{5+0}{68/1.0}\right) = 0.86 < 1.0$$

$$\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} + k_{zy}\frac{M_{y.Ed}+\Delta M_{y.Ed}}{\chi_{LT}M_{y.Rk}/\gamma_{M1}} + k_{zz}\frac{M_{z.Ed}+\Delta M_{z.Ed}}{M_{z.Rk}/\gamma_{M1}} \le 1.0$$

Eq (6.62)

$$\left(\frac{600}{0.81\times1271/1.0}\right)+0.49\times\left(\frac{20+0}{1.0\times68/1.0}\right)+0.81\times\left(\frac{5+0}{68/1.0}\right) = 0.79 < 1.0$$

Both criteria are met, therefore the buckling resistance of the hot finished SHS 150 × 150 × 6.3 in S355 steel is adequate.

5.8 Blue Book Approach

Page references in Section 5.8 are to P363 unless otherwise stated.

The resistances calculated in Sections 5.6 and 5.7 of this example could have been obtained from SCI publication P363.

5.8.1 Design force and moments at ultimate limit state

Design compression force $N_{Ed} = 600$ kN

Design moment about the *y-y* axis (major axis) $M_{y,Ed} = 20$ kNm

Design moment about the *z-z* axis (minor axis) $M_{z,Ed} = 5$ kNm

5.8.2 Cross section classification

$N_{pl,Rd} = 1270$ kN

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$$n = \frac{N_{Ed}}{N_{pl,Rd}}$$

Limiting value of n for Class 2 sections is 1.0

Page D-186

$$n = \frac{600}{1270} = 0.47 < 1.0$$

Therefore, under combined bending and axial compression, the section is at least Class 2.

5.8.3 Cross-sectional resistance

For Class 1 or 2 cross sections there are two verifications that may be performed.

Verification 1 (conservative)

Verify that: — Section 10.2.1

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{pl,Rd}}}+\frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{c,y,Rd}}}+\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{c,z,Rd}}}\leq 1.0$$

As the section is square, $M_{\mathrm{c,y,Rd}}$ & $M_{\mathrm{c,z,Rd}} = M_{\mathrm{c,Rd}}$

For

$$n = \frac{N_{\mathrm{Ed}}}{N_{\mathrm{pl,Rd}}} = \frac{600}{1270} = 0.47$$

$M_{\mathrm{c,Rd}} = 68.2$ kNm — Page D-186

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{pl,Rd}}}+\frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{c,y,Rd}}}+\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{c,z,Rd}}}=\frac{600}{1270}+\frac{20}{68.2}+\frac{5}{68.2}=0.84 < 1.0$$

Therefore this criterion is satisfied.

Verification 2 (more exact)

Verify that:

$$\left(\frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{N,y,Rd}}}\right)^{\alpha}+\left(\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{N,z,Rd}}}\right)^{\beta}\leq 1.0$$

BS EN 1993-1-1
6.2.9.1(6)
Eq (6.41)

From the earlier calculations, — Section 5.6.1 of this example

$\alpha = \beta = 2.21$

For square rolled hollow sections, $M_{\mathrm{N,y,Rd}}$ & $M_{\mathrm{N,z,Rd}} = M_{\mathrm{N,Rd}}$

From interpolation for $n = 0.47$

$M_{\mathrm{N,Rd}} = 47.3$ kNm — Page D-186

$$\left(\frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{N,y,Rd}}}\right)^{\alpha}+\left(\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{N,z,Rd}}}\right)^{\beta}=\left(\frac{20}{47.3}\right)^{2.21}+\left(\frac{5}{47.3}\right)^{2.21}=0.16 < 1.0$$

Therefore the cross-sectional resistance is adequate.

5.8.4 Buckling resistance

When both of the following criteria are satisfied:

- The cross section is Class 1, 2 or 3
- $\gamma_{M1} = \gamma_{M0}$

The buckling verification given in BS EN 1993-1-1 6.3.3 (expressions 6.61 and 6.62) may be simplified to:

$$\frac{N_{Ed}}{N_{b,y,Rd}} + k_{yy}\frac{M_{y.Ed}}{M_{b,Rd}} + k_{yz}\frac{M_{z.Ed}}{M_{c,z,Rd}} \leq 1.0$$

$$\frac{N_{Ed}}{N_{b,z,Rd}} + k_{zy}\frac{M_{y.Ed}}{M_{b,Rd}} + k_{zz}\frac{M_{z.Ed}}{M_{c,z,Rd}} \leq 1.0$$

For the bending moment diagram in Figure 5.3

k_{yy} = 0.81 — Sheet 7

k_{yz} = 0.49 — Sheet 7

k_{zy} = 0.49 — Sheet 7

k_{zz} = 0.81 — Sheet 7

For square rolled hollow sections, $M_{b,Rd} = M_{c,Rd}$, and since $\gamma_{M1} = \gamma_{M0}$, $M_{c,z,Rd} = M_{c,Rd}$

$M_{c,Rd}$ = 68.2 kNm — Page D-186

As the section is a square rolled hollow section and the flexural buckling lengths about both axes are equal, $N_{b,y,Rd} = N_{b,z,Rd} = N_{b,Rd}$.

Since $n < n$ limit i.e. 0.47 < 1.0, the tabulated values of $N_{b,Rd}$ are valid. — Page D-187

From linear interpolation for L_{cr} = 3.5 m

$N_{b,Rd}$ = 1014 kNm

$$\left(\frac{600}{1014}\right) + \left(0.81 \times \frac{20}{68.2}\right) + \left(0.49 \times \frac{5}{68.2}\right) = 0.87 < 1.0$$

$$\left(\frac{600}{1014}\right) + \left(0.49 \times \frac{20}{68.2}\right) + \left(0.81 \times \frac{5}{68.2}\right) = 0.80 < 1.0$$

Therefore, the buckling resistance is adequate.

References are to BS EN 1993-1-1: 2005, including its National Annex, unless otherwise stated.

6 Top chord in a lattice girder

6.1 Scope

The top chord of the lattice girder shown in Figure 6.1 is laterally restrained at locations A, B and C. Verify the adequacy of a hot finished 150 × 150 × 5 SHS in S355 steel for this chord.

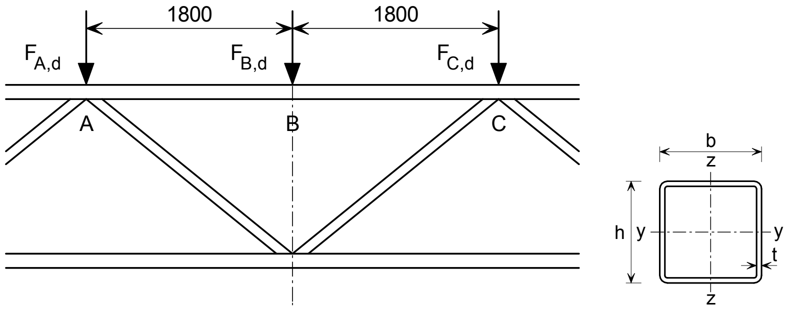

Figure 6.1

The design aspects covered in this example are:

- Cross section classification
- Cross-sectional resistance to combined shear, bending and axial compression
- Buckling resistance for combined bending and axial compression.

The adequacy of the welded joints should be verified using BS EN 1993-1-8. Those verifications are not shown in this example.

6.2 Design values of actions at ultimate limit state

Design concentrated force at A $F_{A,d} = 22.4$ kN
Design concentrated force at B $F_{B,d} = 22.4$ kN
Design concentrated force at C $F_{C,d} = 22.4$ kN

6.3 Design moments and forces at ultimate limit state

From analysis:

Compression force between A and C $N_{Ed} = 525$ kN

The design bending moments and shear force are shown in Figure 6.2.

The design bending moment (M_{Ed}) and corresponding design shear force (V_{Ed}) at B are:

M_{Ed} = 10.1 kNm

V_{Ed} = 11.2 kN

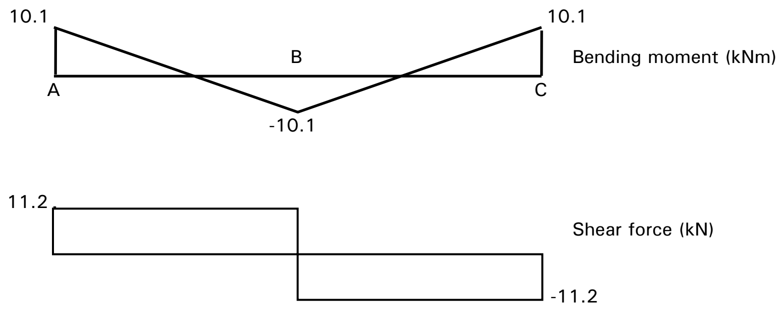

Figure 6.2

6.4 Section properties

For a hot finished 150 × 150 × 5.0 SHS in S355 steel: P363

Depth of section	h	= 150 mm
Width of section	b	= 150 mm
Wall thickness	t	= 5.0 mm
Second moment of area	I	= 1000 cm^4
Radius of gyration	i	= 5.90 cm
Elastic modulus	W_{el}	= 134 cm^3
Plastic modulus	W_{pl}	= 156 cm^3
Area	A	= 28.70 cm^2
Modulus of elasticity	E	= 210000 N/mm^2

3.2.6(1)

For buildings that will be built in the UK, the nominal values of the yield strength (f_y) and the ultimate strength (f_u) for structural steel should be those obtained from the product standard. Where a range is given the lowest nominal value should be used. NA.2.4

For S355 steel and $t \leq 16$ mm

Yield strength $f_y = R_{eH} = 355$ N/mm^2

BS EN 10210-1 Table A.3

6.5 Cross section classification

Table 5.2

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81$$

The elastic stress distribution in a square hollow section under combined bending about one axis and compression can be sketched as shown in Figure 6.3. Therefore the classification of the section may be conservatively determined by using the class limits in the ‘part subject to compression’ section of Table 5.2 given in BS EN 1993-1-1.

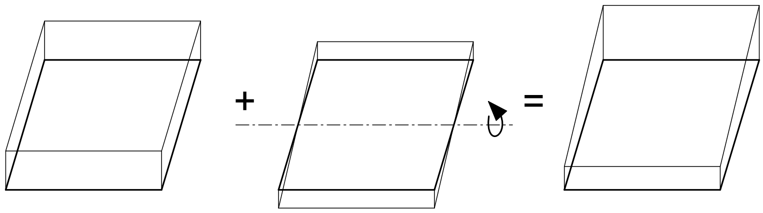

Figure 6.3

Internal compression part (web)

$c = h - 3t = 150 - 3 \times 5 = 135$ mm

$$\frac{c}{t} = \frac{135}{5} = 27$$

The limiting value for Class 1 is $\frac{c}{t} \leq 33\varepsilon = 33 \times 0.81 = 26.73$ — Table 5.2

The limiting value for Class 2 is $\frac{c}{t} \leq 38\varepsilon = 38 \times 0.81 = 30.78$

$26.73 < 27 < 30.78$

Therefore, the cross section is Class 2 under combined bending and compression about the *y-y* axis shown in Figure 6.1.

6.6 Partial factors for resistance

$\gamma_{M0} = 1.0$ — NA.2.15

$\gamma_{M1} = 1.0$

6.7 Cross-sectional resistance

6.7.1 Bending, shear and axial force

At cross section B

If the design shear force (V_{Ed}) is less than 50% of the design plastic shear resistance ($V_{pl,Rd}$), allowance for the shear force on the resistance moment is not required, except where shear buckling reduces the section resistance. — 6.2.10(2)

$$V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}}$$

6.2.6(2) Eq (6.18)

A_v is the shear area and is determined as follows for an SHS.

$$A_v = \frac{Ah}{b+h} = \frac{2870 \times 150}{150 + 150} = 1435.0 \text{ mm}^2$$

6.2.6(3)(f)

$$V_{pl,Rd} = \frac{1435 \times (355/\sqrt{3})}{1.0} \times 10^{-3} = 294 \text{ kN}$$

Design shear force $V_{Ed} = 11.2$ kN

$$\frac{V_{pl,Rd}}{2} = \frac{294}{2} = 147 \text{ kN}$$

11.2 kN < 147 kN

Therefore, the criterion is satisfied, subject to verification of shear buckling.

Verify whether shear buckling reduces the resistance

The shear buckling resistance for webs should be verified according to Section 5 of BS EN 1993-1-5 if: — 6.2.6(6)

$$\frac{h_w}{t_w} > \frac{72\varepsilon}{\eta}$$ — Eq (6.22)

$$h_w = h - 2t = 150 - (2 \times 5) = 140 \text{ mm}$$ — BS EN1993-1-5 Figure 5.1

η may be obtained from BS EN 1993-1-5 or conservatively taken as $\eta = 1.0$

$$t_w = t = 5 \text{ mm}$$

$$\frac{h_w}{t_w} = \frac{140}{5} = 28.0$$

$$\frac{72\varepsilon}{\eta} = \frac{72 \times 0.81}{1.0} = 58.3$$

28.0 < 58.3

Therefore, the shear buckling resistance of the SHS web does not need to be verified.

The effect of the shear force on the resistance to combined bending and axial force does not need to be allowed for.

Combined bending and axial force

For Class 1 and 2 cross sections, verify that: — 6.2.9.1(1) Eq (6.31)

$$M_{Ed} \le M_{N,Rd}$$

where:

$M_{N,Rd}$ is the design plastic moment resistance reduced due to the axial force.

Where fastener holes are not to be accounted for, the design moment resistance for the major axis ($M_{N,y,Rd}$) is determined from: — 6.2.9.1(5)

$$M_{N,y,Rd} = M_{pl,y,Rd} \frac{1-n}{1-0.5a_w} \text{ but } M_{N,y,Rd} \le M_{pl,y,Rd}$$ — Eq (6.39)

The design plastic moment resistance of the cross section about the major axis ($M_{\mathrm{pl,y,Rd}}$) for Class 1 and 2 cross sections is determined from: 6.2.5(2)

$$M_{\mathrm{pl,y,Rd}} = \frac{W_{\mathrm{pl,y}} f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} = \frac{156 \times 10^3 \times 355}{1.0} \times 10^{-6} = 55 \text{ kNm}$$ Eq (6.13)

6.2.9.1(5)

$$n = \frac{N_{\mathrm{Ed}}}{N_{\mathrm{pl,Rd}}}$$

$N_{\mathrm{pl,Rd}}$ is the design plastic resistance of the gross cross section:

$$N_{\mathrm{pl,Rd}} = \frac{A f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} = \frac{2870 \times 355}{1.0} \times 10^{-3} = 1019 \text{ kN}$$ Eq (6.6)

$$n = \frac{525}{1019} = 0.52$$

6.2.9.1(5)

$$a_{\mathrm{w}} = \frac{A - 2bt}{A} \text{ but } a_{\mathrm{w}} \leq 0.5$$

$$a_{\mathrm{w}} = \frac{2870 - (2 \times 150 \times 5)}{2870} = 0.48 < 0.5$$

Therefore, $a_{\mathrm{w}} = 0.48$

$$M_{\mathrm{N,Rd}} = M_{\mathrm{pl,y,Rd}} \frac{1-n}{1-0.5a_{\mathrm{w}}} = 55 \times \frac{1-0.52}{1-(0.5 \times 0.48)} = 35 \text{ kNm}$$ Eq (6.39)

M_{Ed} = 10.1 kNm < 35 kNm

$$\frac{M_{\mathrm{Ed}}}{M_{\mathrm{N,Rd}}} = \frac{10.1}{35} = 0.29 < 1.0$$

Therefore the resistance of the cross section at B to combined bending, shear and axial force is adequate.

By inspection, the resistance of the cross section at A and C is also adequate.

6.8 Buckling resistance of member

6.8.1 Combined bending and axial compression

Cross-sectional resistance at the ends of the member (A and C) to combined bending, shear and axial force is satisfactory (see 6.7.1 above). 6.3.3(2)

For combined bending and axial compression of the member, the following criteria should be satisfied.

$$\frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{y}} N_{\mathrm{Rk}} / \gamma_{\mathrm{M1}}} + k_{\mathrm{yy}} \frac{M_{\mathrm{y,Ed}} + \Delta M_{\mathrm{y,Ed}}}{\chi_{\mathrm{LT}} M_{\mathrm{y,Rk}} / \gamma_{\mathrm{M1}}} + k_{\mathrm{yz}} \frac{M_{\mathrm{z,Ed}} + \Delta M_{\mathrm{z,Ed}}}{M_{\mathrm{z,Rk}} / \gamma_{\mathrm{M1}}} \leq 1.0$$ Eq (6.61)

$$\frac{N_{\mathrm{Ed}}}{\chi_{\mathrm{z}} N_{\mathrm{Rk}} / \gamma_{\mathrm{M1}}} + k_{\mathrm{zy}} \frac{M_{\mathrm{y,Ed}} + \Delta M_{\mathrm{y,Ed}}}{\chi_{\mathrm{LT}} M_{\mathrm{y,Rk}} / \gamma_{\mathrm{M1}}} + k_{\mathrm{zz}} \frac{M_{\mathrm{z,Ed}} + \Delta M_{\mathrm{z,Ed}}}{M_{\mathrm{z,Rk}} / \gamma_{\mathrm{M1}}} \leq 1.0$$ Eq (6.62)

For Class 1, 2 and 3 cross sections $\Delta M_{\mathrm{y,Ed}}$ and $\Delta M_{\mathrm{z,Ed}}$ are zero.

$N_{Rk} = Af_y$ (for Class 1 and 2 cross sections)	Table 6.7
$N_{Rk} = 2870 \times 355 \times 10^{-3} = 1019$ kN	
$M_{Rk} = W_{pl} f_y$ (for Class 1 and 2 cross sections)	Table 6.7
As the section is square	
$M_{z,Rk} = M_{y,Rk} = 156 \times 10^3 \times 355 \times 10^{-6} = 55$ kNm	
Calculation of reduction factors for buckling χ_y, χ_z and χ_{LT}	
For flexural buckling $\chi_y = \chi$	6.3.1.2(1) Eq (6.49)
$\chi = \dfrac{1}{(\Phi + \sqrt{(\Phi^2 - \bar{\lambda}^2)}}$ but $\chi \le 1.0$	
where:	6.3.1.3(1) Eq (6.50)
$\bar{\lambda} = \sqrt{\dfrac{Af_y}{N_{cr}}} = \left(\dfrac{L_{cr}}{i}\right)\left(\dfrac{1}{\lambda_1}\right)$ (For Class 1, 2 and 3 cross sections).	
The buckling lengths may be taken as the distance between restraints, therefore:	
For buckling about *y-y* axis $L_{y,cr} = 3600$ mm	
For buckling about the *z-z* axis $L_{z,cr} = 1800$ mm	
$\lambda_1 = 93.9\varepsilon = 93.9 \times 0.81 = 76.06$	
Buckling about the *y-y* axis:	
$\bar{\lambda}_y = \left(\dfrac{3600}{59.0}\right)\left(\dfrac{1}{76.06}\right) = 0.80$	
$\Phi_y = 0.5\left[1 + \alpha\left(\bar{\lambda}_y - 0.2\right) + \bar{\lambda}_y^{\,2}\right]$	6.3.1.2(1)
For hot finished SHS in S355 steel use buckling curve ‘a’	Table 6.2
For curve ‘a’ the imperfection factor is $\alpha = 0.21$	Table 6.1
$\Phi_y = 0.5 \times \left[1 + 0.21 \times \left(0.80 - 0.2\right) + 0.80^2\right] = 0.88$	6.3.1.2(1)
$\chi_y = \dfrac{1}{(\Phi_y + \sqrt{(\Phi_y^{\,2} - \bar{\lambda}_y^{\,2})}} = \dfrac{1}{0.88 + \sqrt{(0.88^2 - 0.80^2)}} = 0.80$	Eq (6.49)
$0.80 < 1.0$	
Therefore, $\chi_y = 0.80$	
Buckling about the *z-z* axis:	6.3.1.3(1) Eq (6.50)
$\bar{\lambda}_z = \left(\dfrac{1800}{59.0}\right)\left(\dfrac{1}{76.1}\right) = 0.40$	
$\Phi_z = 0.5 \times \left[1 + 0.21 \times \left(0.40 - 0.2\right) + 0.40^2\right] = 0.60$	6.3.1.2(1)

$$\chi_z = \frac{1}{(\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}} = \frac{1}{0.60 + \sqrt{(0.60^2 - 0.40^2)}} = 0.95$$ Eq (6.49)

$0.95 < 1.0$

Therefore,

$\chi_z = 0.95$

Lateral torsional buckling

Square hollow sections are not susceptible to lateral torsional buckling. 6.3.2.1(2)

Therefore, $\chi_{LT} = 1.0$

Calculation of interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} 6.3.3(5)

For sections not susceptible to torsional deformation, use Table B.1 to determine the interaction factors k_{yy}, k_{yz} , k_{zy} and k_{zz}. However, for this example, the design bending moment about the minor axis is zero, therefore values for k_{yz} and k_{zz} are not required.

For class 1 and 2 cross sections

Table B.1

$$k_{yy} = C_{my}\left[1 + \left(\bar{\lambda}_y - 0.20\right)\left(\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right] \quad \text{but}$$

$$k_{yy} \le C_{my}\left[1 + \left(\frac{0.8 N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right]$$

For bending about the *y-y* axis consider points braced in the *z-z* direction. Table B.3

Points A and C are braced in the *z-z* direction. Therefore, the following bending moment diagram between A and C needs to be considered when calculating C_{my}.

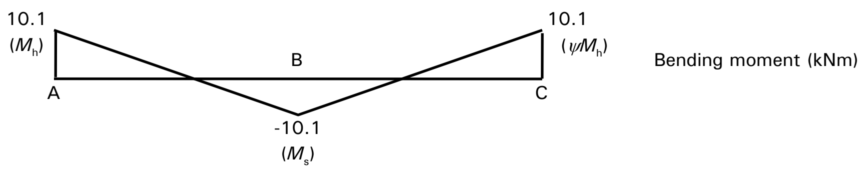

Figure 6.4

From the above bending moment diagram:

$\psi = 1.0$

$$\alpha_s = \frac{M_s}{M_h} = \frac{-10.1}{10.1} = -1.0$$

Therefore,

$C_{my} = -0.8\alpha_s \ge 0.4$

$C_{my} = -0.8 \times -1.0 = 0.8 > 0.4$

Hence,

$C_{my} = 0.8$

$$k_{yy} = C_{my}\left[1+\left(\bar{\lambda}_y - 0.2\right)\left(\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right] \le C_{my}\left[1+\left(\frac{0.8N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right]$$ Table B.1

$$k_{yy} = 0.8\times\left[1+(0.8-0.2)\times\left(\frac{525}{0.8\times1019/1.0}\right)\right] = 1.11$$

$$C_{my}\left[1+\left(\frac{0.8N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right] = 0.8\times\left[1+\left(\frac{0.8\times525}{0.8\times1019/1}\right)\right] = 1.21$$

1.11 < 1.21

Therefore, k_{yy} = 1.11

$$k_{zy} = 0.6k_{yy} = 0.6\times1.11 = 0.67$$ Table B.1

$$\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}} + k_{yy}\frac{M_{y.Ed}+\Delta M_{y.Ed}}{\chi_{LT}M_{y.Rk}/\gamma_{M1}} + k_{yz}\frac{M_{z.Ed}+\Delta M_{z.Ed}}{M_{z.Rk}/\gamma_{M1}} \le 1.0$$ Eq (6.61)

$$\left(\frac{525}{0.8\times1019/1.0}\right) + 1.11\times\left(\frac{10.1+0}{1.0\times55/1.0}\right) + 0 = 0.85 < 1.0$$

$$\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}} + k_{zy}\frac{M_{y.Ed}+\Delta M_{y.Ed}}{\chi_{LT}M_{y.Rk}/\gamma_{M1}} + k_{zz}\frac{M_{z.Ed}+\Delta M_{z.Ed}}{M_{z.Rk}/\gamma_{M1}} \le 1.0$$ Eq (6.62)

$$\left(\frac{525}{0.95\times1019/1.0}\right) + 0.67\times\left(\frac{10.1+0}{1.0\times55/1.0}\right) + 0 = 0.67 < 1.0$$

As both criteria are satisfied, the buckling resistance of the member is adequate.

6.9 Blue Book Approach

Page references in Section 6.9 are to P363 unless otherwise stated.

The resistances calculated in Sections 6.7 and 6.8 of this example could have been obtained from SCI publication P363.

6.9.1 Design forces and moments at ultimate limit state

Axial compression force between A and C N_{Ed} = 525 kN

The design bending moment diagram is shown in Figure 6.2. The design bending moment (M_{Ed}) and corresponding design shear force (V_{Ed}) at B are:

M_{Ed} = 10.1 kNm

V_{Ed} = 11.2 kN

6.9.2 Cross section classification

$N_{pl,Rd}$ = 1020 kN Page D-186

$$n = \frac{N_{Ed}}{N_{pl,Rd}}$$

Limiting value of n for Class 2 sections is 1.0 — Page D-186

$$n = \frac{525}{1020} = 0.51 < 1.0$$

Therefore, under combined bending and axial compression the section is at least Class 2.

6.9.3 Cross-sectional resistance

Shear resistance

$V_{c,Rd} = 294$ kN — Page D-84

$$\frac{V_{Ed}}{V_{c,Rd}} = \frac{11.2}{294} = 0.04 < 1.0$$

Therefore the design shear resistance is adequate.

Compression resistance

$N_{c,Rd} = N_{pl,Rd} = 1020$ kN — Page D-186

$$\frac{N_{Ed}}{N_{c,Rd}} = \frac{525}{1020} = 0.51 < 1.0$$

Therefore the design resistance to compression is adequate.

Bending resistance

$$\frac{V_{c,Rd}}{2} = \frac{294}{2} = 147 \text{ kN}$$

$V_{Ed} = 11.2 \text{ kN} < 147 \text{ kN}$

Therefore the shear at the point of maximum bending moment is low. Thus only the effect of the axial compression on the bending moment resistance needs to be allowed for.

From linear interpolation for $n = 0.51$, — Page D-186

$M_{N,Rd} = 35.7$ kN

$$\frac{M_{y,Ed}}{M_{N,Rd}} = \frac{10.1}{35.7} = 0.28 < 1.0$$

Therefore the design resistance to bending is adequate.

6.9.4 Buckling resistance

Bending and axial compression buckling resistance

When both of the following criteria are satisfied:

- The cross section is Class 1, 2 or 3
- $\gamma_{M1} = \gamma_{M0}$

The verification expressions given in BS EN 1993-1-1 6.3.3 (Expressions 6.61 and 6.62) may be simplified to:

$$\frac{N_{\text{Ed}}}{N_{\text{b,y,Rd}}} + k_{\text{yy}} \frac{M_{\text{y.Ed}}}{M_{\text{b,Rd}}} + k_{\text{yz}} \frac{M_{\text{z.Ed}}}{M_{\text{c,z,Rd}}} \leq 1.0$$

$$\frac{N_{\text{Ed}}}{N_{\text{b,z,Rd}}} + k_{\text{zy}} \frac{M_{\text{y.Ed}}}{M_{\text{b,Rd}}} + k_{\text{zz}} \frac{M_{\text{z.Ed}}}{M_{\text{c,z,Rd}}} \leq 1.0$$

As $M_{\text{z,Ed}} = 0$ kNm the values of k_{yz} and k_{zz} are not required.

For the bending moment diagram in Figure 6.4

$k_{\text{yy}} = 1.11$ — Sheet 7

$k_{\text{zy}} = 0.67$ — Sheet 8

Since $n < n$ limit i.e. $0.51 < 1.0$, the tabulated values of $N_{\text{b,Rd}}$ are valid.

The buckling length for flexural buckling about the major axis (y-y) is 3.6 m

From linear interpolation for $L_{\text{cr}} = 3.6$ m — Page D-187

$N_{\text{b,y,Rd}} = N_{\text{b,Rd}} = 806$ kN

The buckling length for flexural buckling about the minor axis (z-z) is 1.8 m

The shortest buckling length given in SCI P363 is 2 m, therefore for $L_{\text{cr}} = 2.0$ m. — Page D-187

$N_{\text{b,z,Rd}} = N_{\text{b,Rd}} = 959$ kN

For square rolled hollow sections, $M_{\text{b,Rd}} = M_{\text{c,Rd}}$, and since $\gamma_{\text{M1}} = \gamma_{\text{M0}}$:
$M_{\text{c,z,Rd}} = M_{\text{c,Rd}}$

$M_{\text{c,Rd}} = 68.2$ kNm — Page D-186

$$\left(\frac{525}{806}\right) + 1.11 \times \left(\frac{10.1}{68.2}\right) = 0.82 < 1.0$$

$$\left(\frac{525}{959}\right) + 0.67 \times \left(\frac{10.1}{68.2}\right) = 0.65 < 1.0$$

Therefore, the buckling resistance is adequate.

References are to BS EN 1993-1-1: 2005, including its National Annex, unless otherwise stated.

7 Column in simple construction

7.1 Scope

Verify the adequacy of the column shown in Figure 7.1 between levels A and B.

The following assumptions may be made:

- The column is continuous and forms part of a structure of simple construction. (*Simple construction is defined in the Access Steel document SN020.)*
- The column is nominally pinned at the base.
- Beams are connected to the column flange by flexible end plates.

Access Steel document SN020

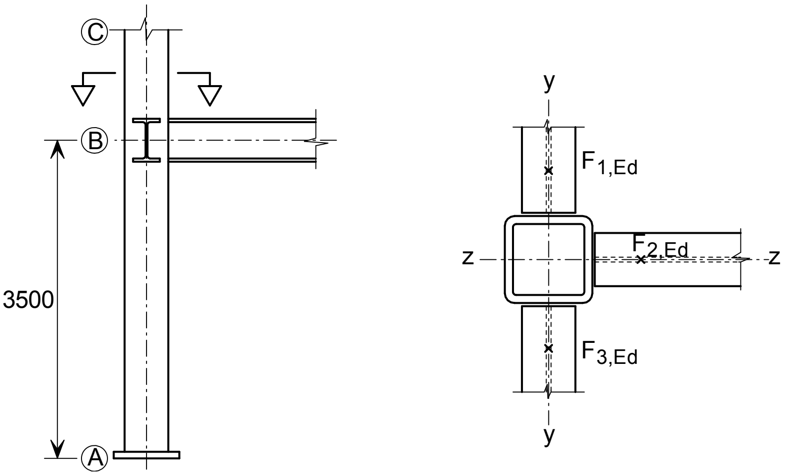

Figure 7.1

The design aspects covered in this example are:

- Cross section classification
- Simplified interaction criteria for combined axial compression and bi-axial bending, as given in the Access Steel document SN048.

7.2 Design values of actions at ultimate limit state

Design force from beam 1 $F_{1,\mathrm{Ed}} = 95$ kN

Design force from beam 2 $F_{2,\mathrm{Ed}} = 130$ kN

Design force from beam 3 $F_{3,\mathrm{Ed}} = 80$ kN

7.3 Design force and moments at ultimate limit state

7.3.1 Compression force

The design compression force in the column between levels B and C is:

$N_{1,\mathrm{Ed}}$ = 165 kN

The design compression force acting on the column between levels A and B is given by:

$$N_{\mathrm{Ed}} = N_{1,\mathrm{Ed}} + F_{1,\mathrm{Ed}} + F_{2,\mathrm{Ed}} + F_{3,\mathrm{Ed}} = 165 + 95 + 130 + 80 = 470 \text{ kN}$$

7.3.2 Moments due to eccentricity

For columns in simple construction the beam reactions are assumed to act at a distance of 100 mm from the face of the column.

Access Steel document SN005

For a hot finished 150 × 150 × 5 SHS in S355 steel.

The design bending moments at level B

$$M_{\mathrm{B,y,Ed}} = F_{2,\mathrm{Ed}}\left(\frac{h}{2} + 100\right) = 130 \times \left(\frac{150}{2} + 100\right) \times 10^{-3} = 22.8 \text{ kNm}$$

$$M_{\mathrm{B,z,Ed}} = \left(F_{1,\mathrm{Ed}} - F_{3,\mathrm{Ed}}\right)\left(\frac{b}{2} + 100\right)$$

$$= \left(95 - 80\right) \times \left(\frac{150}{2} + 100\right) \times 10^{-3} = 2.6 \text{ kNm}$$

These moments are distributed between the column lengths above and below level B in proportion to their bending stiffness. For this purpose the stiffness is defined as the second moment of area about the appropriate axis divided by the storey height. Where the ratio of stiffness does not exceed 1.5, the moment may be shared equally between the columns above and below the joint.

Here the ratio of stiffness is less than 1.5 therefore the moment may be distributed equally between the column lengths above and below the joint. However, to illustrate the other method here the moment has been distributed based on the stiffness of the column above and below the joint. As the section is continuous, the distribution of the moment is determined using storey heights.

Storey A-B height is 3.5 m and Storey B-C height is 3.0 m. Therefore, the design bending moments acting on the column length between levels A and B are:

$$M_{\mathrm{y,Ed}} = 22.8 \times \frac{3}{6.5} = 10.5 \text{ kNm}$$

$$M_{\mathrm{z,Ed}} = 2.6 \times \frac{3}{6.5} = 1.2 \text{ kNm}$$

7.4 Section properties

For a hot finished 150 × 150 × 5 SHS in S355 steel: — P363

Depth of section	h	= 150 mm
Width of section	b	= 150 mm
Wall thickness	t	= 5.0mm
Second moment of area	I	= 1000 cm^4
Radius of gyration	i	= 5.9 cm
Elastic modulus	W_{el}	= 134 cm^3
Plastic modulus	W_{pl}	= 156 cm^3
Area	A	= 28.7 cm^2
Modulus of elasticity	E	= 210000 N/mm^2 — 3.2.6(1)

For buildings that will be built in the UK, the nominal values of the yield strength (f_y) and the ultimate strength (f_u) for structural steel should be those obtained from the product standard. Where a range is given, the lowest nominal value should be used. — NA.2.4

For S355 steel and $t \leq 16$ mm: — BS EN 10210-1 Table A.3

Yield strength $f_y = R_{eH} = 355$ N/mm^2

7.5 Cross section classification

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81$$

Table 5.2

As shown in Example 5, the critical criterion for the hollow section under combined compression and bi-axial bending is the internal part subject to compression.

Internal flange in compression

$$c = b - 3t = 150 - 3 \times 5 = 135 \text{ mm}$$

$$\frac{c}{t} = \frac{135}{5} = 27$$

The limiting value for Class 1 is $\frac{c}{t} \leq 33\varepsilon = 33 \times 0.81 = 26.73$ — Table 5.2

The limiting value for Class 2 is $\frac{c}{t} \leq 38\varepsilon = 38 \times 0.81 = 30.78$

26.73 < 27 < 30.78

Therefore, the cross section is Class 2 under combined biaxial bending and compression.

7.6 Partial factors for resistance

$\gamma_{M0} = 1.0$ — NA.2.15

$\gamma_{M1} = 1.0$

7.7 Simplified interaction criterion

6.3.3(4) of BS EN 1993-1-1 gives two expressions that should be satisfied for members with combined bending and axial compression (see Example 5).

However, for columns in simple construction, the separate verifications for cross section resistance and buckling resistance can be replaced by the interaction criterion given in Access Steel document SN048. This simple criterion may be expressed as:

Access Steel document SN048

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{b,min,Rd}}} + \frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{b,y,Rd}}} + 1.5\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{cb,z,Rd}}} \leq 1.0$$

and may be used when the following are satisfied:

- column is a hot rolled I or H section, or an SHS
- 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-y* and *z-z* directions at each floor level, but is unrestrained between the floors
- The moment ratios (ψ_i) as defined in Table B.3 in BS EN 1993-1-1 are less than the values given in Tables 2.1 or 2.2 in the Access Steel document SN048.
  or
  In the case where a column base is nominally pinned (i.e. $\psi_{\mathrm{y}} = 0$ and $\psi_{\mathrm{z}} = 0$) the axial force ratio satisfies the following limit:
  $$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{b,y,Rd}}} \leq 0.83$$

Here the

- Section is a Class 2 hot finished SHS
- Moment ratios are $\psi_{\mathrm{y}} = 0$ and $\psi_{\mathrm{z}} = 0$, as the base of the column is nominally pinned (see Figure 7.2). Therefore determine the axial force ratio.

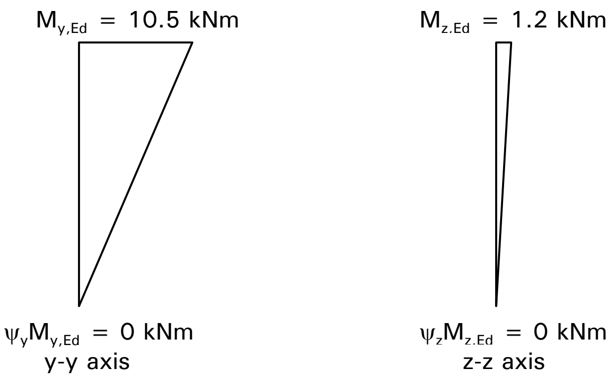

Figure 7.2

Axial force ratio:

$$N_{\mathrm{b,y,Rd}} = \frac{\chi_{\mathrm{y}} A f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}}$$

Access Steel document SN048

$$\chi_{\mathrm{y}} = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}_y^{\,2}}}$$

6.3.1.3 Eq (6.49)

$$\bar{\lambda}_{\mathrm{y}} = \sqrt{\frac{A f_{\mathrm{y}}}{N_{\mathrm{cr}}}} = \frac{L_{cr}}{i} \times \frac{1}{\lambda_1}$$

6.3.1.3 Eq (6.50)

$$\lambda_1 = 93.9 \times \varepsilon = 93.9 \times 0.81 = 76.1$$

6.3.1.3

Buckling lengths about both axes:

$$L_{\mathrm{cr}} = L = 3500 \text{ mm}$$

$$\bar{\lambda}_{\mathrm{y}} = \frac{L_{cr}}{i} \cdot \frac{1}{\lambda_1} = \frac{3500}{59} \times \frac{1}{76.1} = 0.78$$

6.3.1.3 Eq (6.50)

For hot finished hollow sections in S355 steel use buckling curve a — Tables 6.1

For buckling curve a the imperfection factor is $\alpha = 0.21$ — Tables 6.2

$$\Phi = 0.5\left[1 + a(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right]$$

6.3.1.2

$$= 0.5 \times \left[1 + 0.21 \times (0.78 - 0.2) + 0.78^2\right] = 0.87$$

$$\chi_{\mathrm{y}} = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}_y^{\,2}}} = \frac{1}{0.87 + \sqrt{0.87^2 - 0.78^2}} = 0.80$$

6.3.1.3 Eq (6.49)

$$N_{\mathrm{b,y,Rd}} = \frac{\chi_{\mathrm{y}} A f_{\mathrm{y}}}{\gamma_{\mathrm{M1}}} = \frac{0.8 \times 2870 \times 355 \times 10^{-3}}{1.0} = 815 \text{ kN}$$

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{b,y,Rd}}} = \frac{470}{815} = 0.58$$

$0.58 < 0.83$

Therefore all the criteria given above are met, so the simplified expression may be used for this example.

Simplified interaction criterion:

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{b,min,Rd}}} + \frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{b,y,Rd}}} + 1.5\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{cb,z,Rd}}} \le 1.0$$

where:

$N_{\mathrm{b,min,Rd}}$ is the lesser of $N_{\mathrm{b,y,Rd}}$ and $N_{\mathrm{b,z,Rd}}$ in this example $N_{\mathrm{b,y,Rd}}$ and $N_{\mathrm{b,z,Rd}}$ are equal as the section is square and the buckling lengths are the same for both axes.

Therefore, $N_{\mathrm{b,min,Rd}} = N_{\mathrm{b,y,Rd}}$

$$N_{b,y,Rd} = \frac{\chi_y A f_y}{\gamma_{M1}} = 815 \text{ kN}$$

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

$$M_{cb,z,Rd} = \frac{f_y W_{pl,z}}{\gamma_{M1}}$$

Because square hollow sections are not susceptible to lateral torsional buckling,

$$\chi_{LT} = 1.0$$

$$M_{b,y,Rd} = 1.0 \times \frac{355 \times 156 \times 10^3}{1.0} \times 10^{-6} = 55 \text{ kNm}$$

In this example γ_{M1} has the same value as γ_{M0} therefore;

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$$M_{cb,z,Rd} = M_{pl,z,Rd} = \frac{f_y W_{pl,z}}{\gamma_{M0}} = \frac{355 \times 156 \times 10^3}{1.0} \times 10^{-6} = 55 \text{ kNm}$$

$$\frac{N_{Ed}}{N_{b,min,Rd}} + \frac{M_{y,Ed}}{M_{b,y,Rd}} + 1.5 \frac{M_{z,Ed}}{M_{cb,z,Rd}} = \frac{470}{815} + \frac{10.5}{55} + 1.5 \times \left(\frac{1.2}{55}\right) = 0.80 < 1.0$$

Therefore, the resistance of the hot finished 150 × 150 × 5 SHS in S355 steel is adequate.

7.8 Blue Book Approach

Page references in Section7.8 are to P363 unless otherwise stated.

The resistances calculated in Section 7.7 of this example could have been obtained from SCI publication P363.

7.8.1 Forces and moment at ultimate limit state

Design compression force in column between levels A and B is:

$$N_{Ed} = 470 \text{ kN}$$

The design bending moments at B in storey A–B due to the reactions of the floor beams are:

$$M_{y,Ed} = 10.5 \text{ kNm}$$

$$M_{z,Ed} = 1.2 \text{ kNm}$$

7.8.2 Cross section classification

$$N_{pl,Rd} = 1020 \text{ kN}$$

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$$n = \frac{N_{Ed}}{N_{pl,Rd}}$$

Limiting value of n for Class 2 sections is 1.0

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$$N = \frac{470}{1020} = 0.46 < 1.0$$

Therefore, under combined axial compression and bending the section is at least Class 2.

7.8.3 Simplified interaction criterion

The simplified interaction criterion given in Access Steel document SN048 is used here to verify the adequacy of the member.

Simplified interaction criterion:

Access Steel document SN048

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{b,min,Rd}}} + \frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{b,y,Rd}}} + 1.5\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{cb,z,Rd}}} \leq 1.0$$

$N_{\mathrm{b,min,Rd}}$ is the lesser of $N_{\mathrm{b,y,Rd}}$ and $N_{\mathrm{b,z,Rd}}$ in this example $N_{\mathrm{b,y,Rd}}$ and $N_{\mathrm{b,z,Rd}}$ are equal as the section is square and the buckling lengths are the same for both axes, therefore,

$$N_{\mathrm{b,min,Rd}} = N_{\mathrm{b,y,Rd}} = N_{\mathrm{b,z,Rd}} = N_{\mathrm{b,Rd}}$$

Since $n < n$ limit i.e. $0.47 < 1.0$, the tabulated values of $N_{\mathrm{b,Rd}}$ are valid.

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From linear interpolation for $L_{\mathrm{cr}} = 3.5$ m

$N_{\mathrm{b,Rd}} = 818$ kNm

For square rolled hollow sections, $M_{\mathrm{b,y,Rd}} = M_{\mathrm{c,Rd}}$

As $\gamma_{\mathrm{M1}} = \gamma_{\mathrm{M0}}$, $M_{\mathrm{cb,z,Rd}} = M_{\mathrm{c,Rd}}$

$M_{\mathrm{c,Rd}} = 55.4$ kNm

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Therefore,

$$\frac{N_{\mathrm{Ed}}}{N_{\mathrm{b,min,Rd}}} + \frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{b,y,Rd}}} + 1.5\frac{M_{\mathrm{z,Ed}}}{M_{\mathrm{cb,z,Rd}}} = \frac{470}{818} + \frac{10.5}{55.4} + 1.5 \times \left(\frac{1.2}{55.4}\right) = 0.80 < 1.0$$

Therefore, the simplified criterion is verified and thus the resistance of the section is adequate.

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BS EN 1991-1-6	Part 1-6: General actions. Actions during execution
BS EN 1991-1-7	Part 1-7: General actions. Accidental actions

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SN020 NCCI: "Simple Construction" - concept and typical frame arrangements

SN048 NCCI: Verification of columns in simple construction – a simplified interaction criterion (GB). (*This is a localized resource for UK*)

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