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Steelwork design guide to BS 5950-1:2000 Volume 2: Worked examples

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FOREWORD

This publication replaces an earlier SCI publication (P002) that provided a series of worked examples using the previous issue of the design code for steelwork in buildings (BS 5950-1:1990). A revised design code, (BS 5950-1:2000), which incorporated significant technical revisions, came into effect in 2001 and this led to the need to update those earlier examples.

The publication is a companion to the 'Blue Book', *Steelwork design to BS 5950-1: 2000 – Volume 1*, which provides section property and member capacity tables. Further guidance on the application of the Code can be found in *Introduction to steelwork design to BS 5950-1:2000* (P325).

The present publication has been prepared by Dr Martin Heywood and Dr James Lim of The Steel Construction Institute and presents many new examples as well as revised versions of some of the examples in the earlier publication.

Further advice and guidance was received during the drafting of this publication from Mr Abdul Malik, Mr Charles King and Mr Thomas Cosgrove, all of The Steel Construction Institute.

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INTRODUCTION

This publication is intended to provide practising engineers and students with a guide to meeting the requirements of BS 5950-1: 2000 *Structural use of steelwork in building*[1] and contains worked examples that have been prepared to give a detailed indication of the process of designing members to this Code.

The worked examples deal with all the main checks required by the Code and point out those which will normally be critical in design. The emphasis has been to illustrate the points in the Code rather than attempt to match practical cases exactly.

In addition to the design of simple elements, a number of frames and combinations of members have been included. Generally, the solutions illustrated are aimed at the most economical use of steel but it is emphasized that other solutions may be equally acceptable. No consideration has been given to factors governing erection and fabrication; the considerations of these factors and the standardization of sizes may well lead to solutions with better overall economy.

Unless otherwise stated, the clause and table numbers given in the right-hand margin of the examples refer to BS 5950-1:2000[1]. Where a reference is made to a page in “Vol 1” this refers to *Steelwork design to BS 5950-1:2000 – Volume 1 Section properties member capacities*[2], commonly known as the “Blue Book”.

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CALCULATION SHEET

Job No. CDS 153		Sheet 1 of 1	Rev
Job Title Example no. 1			
Subject Choosing a steel sub-grade			
Client SCI	Made by MDH	Date Jun 2003	
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1 Choosing a steel sub-grade

1.1 Introduction

An exposed steel structure is proposed. The steel is S355 to BS EN 10025 and the thickest element is 30 mm. Beams are welded to the column flanges and the maximum tensile stress is 200 N/mm^2. Choose an appropriate sub-grade to avoid brittle fracture.

1.2 Sub-grade selection

Basic requirement: $t \leq Kt_1$ 2.4.4

Since the maximum thickness t = 30 mm, the steel sub-grade should be chosen such that

$t_1 \geq 30/K$

The nominal yield strength Y_{nom} = 355 N/mm^2
The maximum tensile stress in the component = 200 N/mm^2
Therefore, stress > 0.3 Y_{nom}

For welded connections to unstiffened flanges with stress > 0.3 Y_{nom} Table 3

$K = 0.5$

The requirement for t_1 is therefore

$t_1 \geq 60$ mm

For BS EN 10025 - S355 K2 in external conditions (–15°C), t_1 = 66 mm. Table 4

Therefore, $Kt_1 = 0.5 \times 66 = 33$ mm

30 mm < 33 mm **OK**

Finally, check that the maximum thickness in the component does not exceed the limit for which the full Charpy impact value applies, t_2, as given in Table 6. Table 6

Basic requirement: $t < t_2$

For all "sections" of grade S355 to BS EN 10025, t_2 = 100 mm.

30 mm < 100 mm **OK**

<u>Adopt BS EN 10025 - S355 K2</u>

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CALCULATION SHEET

Job No. CDS 153		Sheet 1 of 7	Rev
Job Title Example no. 2			
Subject Simply supported restrained beam			
Client SCI	Made by MDH	Date Jun 2003	
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2 Simply supported restrained beam

2.1 Introduction

The beam shown in Figure 2.1 is fully restrained along its length and has stiff bearing of 50 mm at the supports and 75 mm under the point load. Design the beam in S275 steel for the loading shown below.

Two solutions are presented: the full calculation in Sections 2.2 and 2.3 and the simplified approach using Volume 1[2] (the "Blue Book" approach) in Section 2.4.

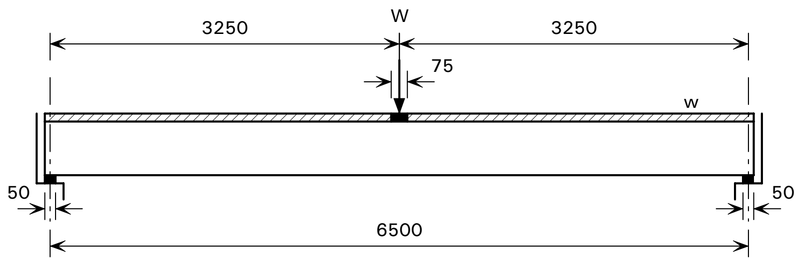

Figure 2.1

2.1.2 Loading (unfactored)

Dead loads:
Distributed load (including s/w) w_d = 15 kN/m
Point load W_d = 40 kN

Imposed loads:
Distributed load w_i = 30 kN/m
Point load W_i = 50 kN

2.1.3 Load Factors

Dead load factor γ_{fd} = 1.4 — Table 2
Imposed load factor γ_{fi} = 1.6

2.1.4 Factored loads

$w' = w_d\,\gamma_d + w_i\,\gamma_{fi} = (15 \times 1.4) + (30 \times 1.6) = 69$ kN/m

$W' = W_d\,\gamma_{fd} + W_i\,\gamma_{fi} = (40 \times 1.4) + (50 \times 1.6) = 136$ kN

2.1.5 Maximum moment and shear

Maximum moment occurs at the centre:

$$M = \frac{w'L^2}{8} + \frac{W'L}{4} \qquad M = \frac{69 \times 6.5^2}{8} + \frac{136 \times 6.5}{4} = 585 \text{ kNm}$$

Example 2 Simply supported beam	Sheet 2 of 7	Rev

Shear force at the ends:

$$F_{ve} = \frac{w'L}{2} + \frac{W'}{2} \qquad F_{ve} = \frac{69 \times 6.5}{2} + \frac{136}{2} \qquad = 292 \text{ kN}$$

Shear force at the centre:

$$F_{vc} = F_{ve} - \frac{w'L}{2} \qquad F_{vc} = 292 - \frac{69 \times 6.5}{2} \qquad = 67.8 \text{ kN}$$

2.2 Member checks

2.2.1 Trial section

The initial trial section is selected to give a suitable moment capacity. This initial member size is then checked to ensure its suitability in all other respects.

Try 533 x 210 x 92 UB in grade S275

From member capacity tables:			Vol 1
Moment capacity	M_c	= 649 kNm	Page C-59
From section property tables:			
Depth	D	= 533.1 mm	Vol 1
Width	B	= 209.3 mm	Page B-2
Web thickness	t	= 10.1 mm	
Flange thickness	T	= 15.6 mm	
Depth between fillets	d	= 476.5 mm	
Plastic modulus	S_x	= 2360 cm^3	Vol 1
Elastic modulus	Z_x	= 2070 cm^3	Page B-3
Local buckling ratios:			
Flange	b/T	= 6.71	Vol 1
Web	d/t	= 47.2	Page B-2

2.2.2 Classify the cross section

Grade of steel = S275

T < 16 mm — 3.1.1

Therefore p_y = 275 N/mm^2 — Table 9

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{275}} = 1.0$$

For the outstand element of a compression flange, the limiting b/T for a class 1 plastic flange is 9ε. — 3.5.2 Table 11

Limiting b/T = 9ε = 9.0

The actual b/T = 6.71 < 9.0

Therefore, **the flange is class 1**.

Since the section is symmetrical and subject to pure bending, the neutral axis is at mid depth. For this case, the limiting d/t for a class 1 plastic web is 80ε. — 3.5.2 Table 11

Limiting d/t = 80ε = 80.0

The actual d/t = 47.2 < 80.0

Therefore, **the web is class 1**.

The flange and the web are both class 1, therefore, **the cross section is class 1**.

Example 2 Simply supported beam	Sheet 3 of 7	Rev

2.2.3 Check for shear buckling

If the d/t ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000[1]. 4.2.3

$70\varepsilon = 70.0$

In this case, $d/t = 47.2 < 70\varepsilon$, so **there is no need to check for shear buckling**.

2.2.4 Check the shear capacity

Basic requirement $F_v \leq P_v$ 4.2.3

$P_v = 0.6\, p_y A_v$

$p_y = 275$ N/mm^2

$A_v = tD = 10.1 \times 533.1 = 5384$ mm^2

$P_v = 0.6 \times 275 \times 5384 \times 10^{-3} = 888$ kN

At the ends of the member $F_v = 292$ kN

292 kN < 888 kN

Therefore, **the shear capacity is adequate.**

2.2.5 Check the moment capacity

Basic requirement $M_x \leq M_{cx}$

Check whether the shear is "high" (i.e. $F_v > 0.6\ P_v$) or "low" at the point of maximum moment. 4.2.5.2

At the centre of the member, $F_{vc} = 67.8$ kN

$0.6\ P_v = 0.6 \times 888 = 533$ kN

67.8 kN < 533 kN

Therefore, **the shear is low**.

For low shear, the moment capacity for a class 1 section is given by: 4.2.5.2

$M_{cx} = p_y\, S_x$

$M_{cx} = 275 \times 2360 \times 10^{-3} = 649$ kNm

Check limit to avoid irreversible deformation under serviceability loads. 4.2.5.1

For a simply supported beam $M_{cx} \leq 1.2\, p_y Z_x$

$1.2 p_y Z_x = 1.2 \times 275 \times 2070 \times 10^{-3} = 683$ kNm

Therefore $M_{cx} = 649$ kNm

From above, $M_x = 585$ kNm

585 kNm < 649 kNm

Therefore, **the moment capacity is adequate.**

Note:

1. It will be found that in most cases there is no need to reduce the moment capacity to allow for shear when using rolled I- or H-sections.
2. Except for a few heavy universal column sections, $M_c = p_y S_x$ will usually be the governing criterion for the moment capacity.

Example 2 Simply supported beam	Sheet 4 of 7	Rev

2.2.6 Web bearing and buckling under the point load

Bearing capacity of the unstiffened web

Basic requirement $F_x \le P_{bw}$

$P_{bw} = (b_1 + nk)\, t\, p_{yw}$ — 4.5.2.1

$b_1 = 75$ mm

$n = 5.0$ (not at the end of the member)

$k = T + r = 15.6 + 12.7 = 28.3$ mm

$(b_1 + nk) = 75 + (5.0 \times 28.3) = 216.5$ mm

$P_{bw} = 216.5 \times 10.1 \times 275 \times 10^{-3} = 601$ kN

$F_x = 136$ kN

136 kN < 601 kN

Therefore, **the bearing capacity of the unstiffened web under the point load is adequate.**

Buckling resistance of the unstiffened web

Basic requirement $F_x \le P_x$ — 4.5.3.1

$$P_x = \frac{25\,\varepsilon\, t}{\sqrt{(b_1 + nk)\, d}} P_{bw}$$

$\varepsilon = 1.0$

$$P_x = \frac{25 \times 1.0 \times 10.1}{\sqrt{216.5 \times 476.5}} \times 601 = 472 \text{ kN}$$

$F_x = 136$ kN

136 kN < 472 kN

Therefore, **the buckling resistance of the unstiffened web under the point load is adequate.**

2.2.7 Web bearing and buckling at the support

Bearing capacity of the unstiffened web

Basic requirement $F_x \le P_{bw}$ — 4.5.2.1

$P_{bw} = (b_1 + nk)\, t\, p_{yw}$

At the end of a member, $n = 2 + 0.6\, b_e / k$ but ≤ 5,

$b_e = 0$ mm

$b_1 = 50$ mm

$k = T + r = 15.6 + 12.7 = 28.3$ mm

$$n = 2 + \frac{0.6 \times 0}{28.3} = 2.0$$

$(b_1 + nk) = 50 + (2.0 \times 28.3) = 106.6$ mm

$P_{bw} = 106.6 \times 10.1 \times 275 \times 10^{-3} = 296$ kN

$F_x = 292$ kN

292 kN < 296 kN

Therefore, **the bearing capacity of the unstiffened web at the support is adequate.**

Example 2 Simply supported beam	Sheet 5 of 7	Rev

Buckling resistance of the unstiffened web

Basic requirement $F_x \le P_x$ — 4.5.3.1

Check whether $a_e < 0.7d$,

$a_e = 25$ mm

$0.7d = 0.7 \times 476.5 = 333.6$ mm

Since $a_e < 0.7d$, the buckling resistance P_x is given by

$$P_x = \frac{a_e + 0.7d}{1.4d} \frac{25\varepsilon t}{\sqrt{(b_1 + nk)d}} P_{bw}$$

$$= \frac{25 + 333.6}{1.4 \times 476.5} \times \frac{25 \times 1.0 \times 10.1}{\sqrt{106.6 \times 476.5}} \times 296 \quad = 178 \text{ kN}$$

$F_x = 292$ kN

292 kN > 178 kN

The buckling resistance of the web at the support is NOT adequate and a load-carrying stiffener must be provided at this location.

2.2.8 SLS deflection check

In addition to checking the adequacy of the beam at ULS, it is also necessary to check the maximum deflection under working loads. — 2.5.2

The serviceability loads are taken as the unfactored imposed loads, i.e.:

Distributed load $w_i = 30$ kN/m

Point load $W_i = 50$ kN

$E = 205$ kN/mm^2 — 3.1.3 Vol 1 Page B-3

$I = 55200$ cm^4

The total deflection δ is given by:

$$\delta = \frac{1}{EI}\left[\frac{5w_i L^4}{384} + \frac{W_i L^3}{48}\right] = \frac{1}{205 \times 55200}\left[\frac{5 \times 30 \times 6.5^4}{384} + \frac{50 \times 6.5^3}{48}\right] \times 10^5$$

$= 8.69$ mm

Assume that the beam carries a plaster finish. — Table 8

The suggested limit in BS 5950-1:2000[1] is span/360.

$$\frac{L}{360} = \frac{6500}{360} = 18.1 \text{ mm.}$$

8.69 mm < 18.1 mm

The deflection is satisfactory.

<u>Adopt 533 x 210 x 92 UB in grade S275</u>

(with load carrying stiffeners at the supports).

2.3 Blue Book approach

The member capacities calculated in section 2.2 of this example could have been obtained directly from Volume 1[2].

Try 533 x 210 x 92 UB in grade S275

2.3.1 Shear capacity

Vol 1
Page C-79

$P_v = 888$ kN
$F_v = 292$ kN
292 kN < 888 kN
Therefore, **the shear capacity is adequate.**

2.3.2 Moment capacity

Vol 1
Page C-59

$M_{cx} = 649$ kNm
$M_x = 585$ kNm
585 kNm < 649 kNm
Therefore, **the moment capacity is adequate.**

2.3.3 Web bearing capacity

$P_w = C_1 + b_1 C_2$

At the location of the point load

Vol 1
Page C-79

$C_1 = 393$ kN
$C_2 = 2.78$ kN/mm
$b_1 = 75$ mm
$P_w = 393 + (2.78 \times 75) = 601$ kN
$F_x = 136$ kN
136 kN < 601 kN
Therefore, **the bearing capacity of the unstiffened web under the point load is adequate.**

At the support

Vol 1
Page C-79

$C_1 = 157$ kN
$C_2 = 2.78$ kN/mm
$b_1 = 50$ mm
$P_w = 157 + (2.78 \times 50) = 296$ kN
$F_x = 292$ kN
292 kN < 296 kN
Therefore, **the bearing capacity of the unstiffened web at the support is adequate.**

2.3.4 Web buckling resistance

$$P_x = K(C_4 P_w)^{0.5}$$

At the location of the point load

Vol 1
Page C-79

$K = 1$
$C_4 = 372$ kN
$P_w = 601$ kN
$P_x = (372 \times 601)^{0.5} = 473$ kN
$F_x = 136$ kN
136 kN < 473 kN
Therefore, **the buckling resistance of the unstiffened web under the point load is adequate.**

Example 2 Simply supported beam	Sheet 7 of 7	Rev

At the support

$$\boldsymbol{K} = \frac{a_e}{1.4d} + 0.5 \quad \text{but} \le 1$$

Vol 1 Page C-79

$1.4d = 667$ mm

$$\boldsymbol{K} = \frac{25}{667} + 0.5 \quad = 0.537$$

$C_4 = 372$ kN

$P_w = 296$ kN

$P_x = 0.537 \times (372 \times 296)^{0.5} \quad = 178$ kN

$F_x = 292$ kN

292 kN > 178 kN

The buckling resistance of the web at the support is NOT adequate and a load-carrying stiffener must be provided at this location.

Note

Volume 1 of the Blue Book does not include deflection values, so the SLS deflection check must be carried out as in Section 2.2.8 of this example.

Adopt 533 x 210 x 92 UB in grade S275

(with load carrying stiffeners at the supports).

The Steel Construction Institute	Job No. CDS 153	Sheet 1 of 6	Rev
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 623345 Fax: (01344) 622944	Job Title Example no. 3		
	Subject Unrestrained beam with end moments		
CALCULATION SHEET	Client SCI	Made by MDH	Date Jun 2003
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3 Unrestrained beam with end moments

3.1 Introduction

The beam shown in Figure 3.1 has end moments of 282 kNm and 231 kNm (due to the factored loads). The intermediate point loads are applied through the bottom flange as shown below. These point loads **do not** provide restraint against lateral-torsional buckling. Design the beam in S275 steel.

Two solutions are presented: the full calculation and the simplified approach using Volume 1[2] (the "Blue Book" approach).

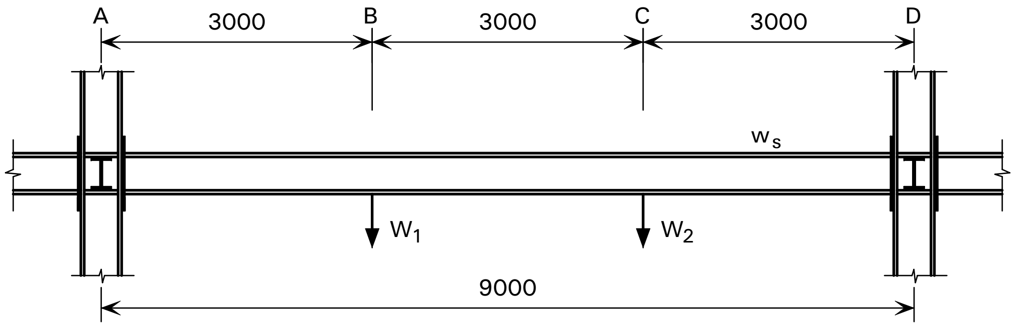

Figure 3.1

3.1.2 Loading (unfactored)

Dead loads:

Self-weight	w_s	= 3 kN/m
Point load	W_{1d}	= 40 kN
Point load	W_{2d}	= 20 kN

Imposed loads:

Point load	W_{1i}	= 60 kN
Point load	W_{2i}	= 30 kN

3.1.3 Load Factors

Table 2

Dead load factor γ_{fd} = 1.4

Imposed load factor γ_{fi} = 1.6

3.1.4 Factored loads

$w_s' = w_s\,\gamma_{fd} = (3 \times 1.4) = 4.2$ kN/m

$W_1' = W_{1d}\,\gamma_{fd} + W_{1i}\,\gamma_{fi} = (40 \times 1.4) + (60 \times 1.6) = 152$ kN

$W_2' = W_{2d}\,\gamma_{fd} + W_{2i}\,\gamma_{fi} = (20 \times 1.4) + (30 \times 1.6) = 76$ kN

The design shear forces and bending moments due to factored loads are as shown in Figure 3.2.

Example 3 Unrestrained beam with end moments	Sheet 2 of 6	Rev

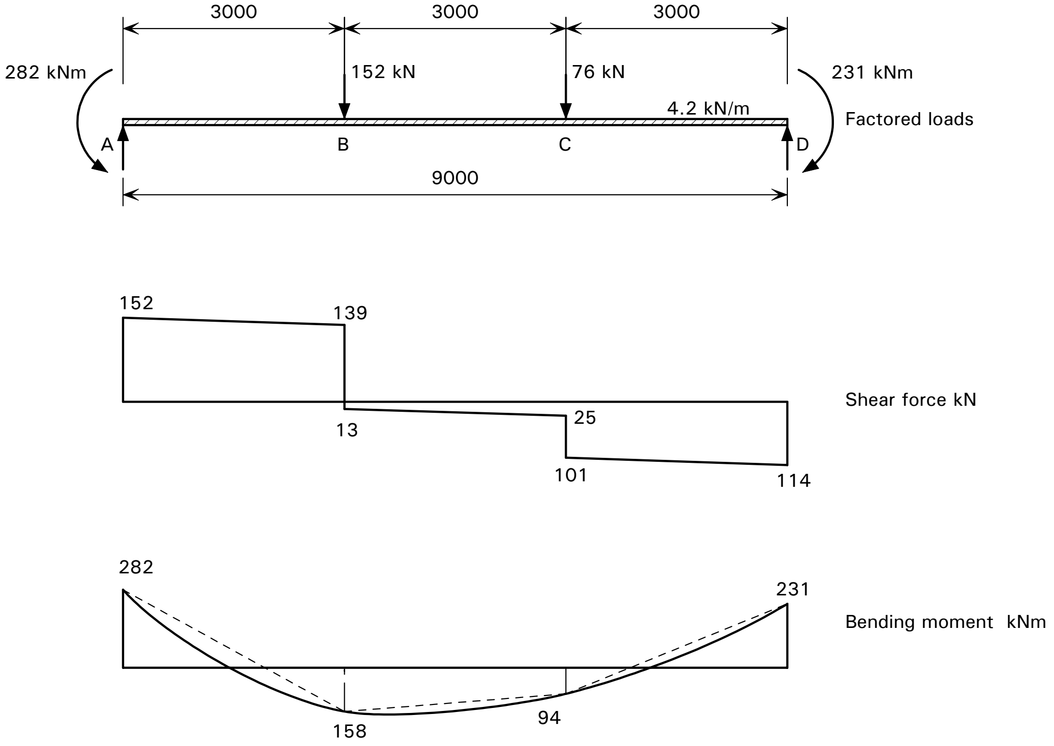

Figure 3.2

3.2 Member checks

3.2.1 Determine the effective length

Since the beam is unrestrained between the supports, there is only one segment to consider in this example, with a length equal to the beam length.

At the supports the following conditions exist:

Beam torsionally restrained.
Compression flange laterally restrained.
Both flanges fully restrained against rotation

Therefore, $L_E = 0.7L = 0.7 \times 9000 = 6300$ mm. — 4.3.5 Table 13

Note: In practice $0.7L$ might be difficult to achieve (not easy to guarantee full rotational restraint) and designers might choose $0.85L$ instead.

3.2.2 Calculate m_{LT}

In BS 5950-1:2000[1] the m-factor method is used for all cases, even when there is loading applied between the restraints, as in this example. Since there is loading between the restraints, it is necessary to calculate m_{LT} using the following general formula:

$$m_{LT} = 0.2 + \frac{0.15M_2 + 0.5M_3 + 0.15M_4}{M_{max}} \quad \text{but } m_{LT} \geq 0.44$$

4.3.6.6 Table 18

The moments M_2 and M_4 are the values at the quarter points, the moment M_3 is the value at mid-length and M_{max} is the maximum moment in the segment. The distributed load is only a small part of the total loading, so the bending moment diagram can be approximated to a series of straight lines as shown in Figure 3.3.

Example 3 Unrestrained beam with end moments	Sheet 3 of 6	Rev

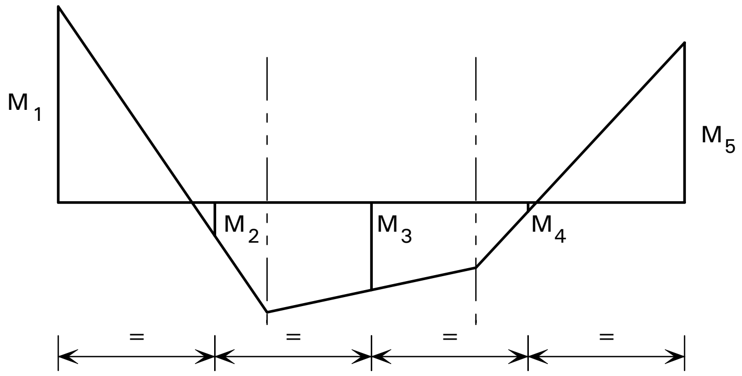

Figure 3.3 *Bending moment*

M_2 = 48 kNm, M_3 = 126 kNm, M_4 = 13 kNm, M_{max} = M_1 = 282 kNm

$$m_{LT} = 0.2 + \frac{(0.15 \times 48) + (0.5 \times 126) + (0.15 \times 13)}{282} = 0.46$$

3.2.3 Trial section

The initial trial section is selected to give a suitable moment capacity and resistance to lateral-torsional buckling. This initial member size is then checked to ensure its suitability in all other respects.

For an unrestrained beam, the maximum major axis moment M_x must satisfy the following conditions:

$M_x \leq M_{cx}$ and $M_x \leq M_b / m_{LT}$ — 4.3.6.2

The chosen beam must, therefore, satisfy:
M_{cx} ≥ 282 kNm and
$M_b \geq 282 \times 0.46$ = 130 kNm for an effective length of 6.3 m.

Try 457 x 191 x 67 UB in grade S275
From member capacity tables:

Moment capacity	M_c	= 405 kNm	Vol 1
Buckling resistance	M_b	= 159 kNm for L_E = 6.0 m	Page C-59
Buckling resistance	M_b	= 133 kNm for L_E = 7.0 m	

From section property tables:

Depth	D	= 453.4 mm	Vol 1
Width	B	= 189.9 mm	Page B-4
Web thickness	t	= 8.5 mm	
Flange thickness	T	= 12.7 mm	
Depth between fillets	d	= 407.6 mm	
Plastic modulus	S_x	= 1470 cm^3	Vol 1
Elastic modulus	Z_x	= 1300 cm^3	Page B-5
Radius of gyration	r_y	= 4.12 cm	
Buckling parameter	u	= 0.872	
Torsional index	x	= 37.9	

Local buckling ratios:

Flange	b/T	= 7.48	Vol 1
Web	d/t	= 48.0	Page B-4

Example 3 Unrestrained beam with end moments	Sheet 4 of 6	Rev

3.2.4 Classify the cross section

Grade of steel = S275 (3.1.1)
T < 16 mm (Table 9)
Therefore p_y = 275 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{275}} = 1.0$$

For the outstand element of a compression flange, the limiting b/T for a class 1 plastic flange is 9ε. Limiting b/T = 9ε = 9.0 (3.5.2, Table 11)
The actual b/T = 7.48 < 9.0
Therefore, **the flange is class 1**.

Since the section is symmetrical and subject to pure bending, the neutral axis is at mid depth. For this case, the limiting d/t for a class 1 plastic web is 80ε. (3.5.2, Table 11)
Limiting d/t = 80ε = 80.0
The actual d/t = 48.0 < 80.0
Therefore, the **web is class 1**.
The flange and the web are both class 1, therefore, **the cross section is class 1**.

3.2.5 Check for shear buckling

If the d/t ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000[1]. (4.2.3)
70ε = 70.0
In this case, d/t = 48.0 < 70ε, so **there is no need to check for shear buckling**.

3.2.6 Check the shear capacity

Basic requirement $F_v \le P_v$ (4.2.3)
P_v = $0.6\, p_y A_v$
p_y = 275 N/mm^2
A_v = tD = 8.5 × 453.4 = 3854 mm^2
P_v = 0.6 × 275 × 3854 × 10^{-3} = 636 kN
The highest shear force occurs at A, where F_v = 152 kN
152 kN < 636 kN
Therefore, **the shear capacity is adequate.**

3.2.7 Check the moment capacity

Basic requirement $M_x \le M_{cx}$
Check whether the shear is "high" (i.e. F_v > 0.6 P_v) or "low" at the point of maximum moment. The maximum moment occurs at A, where F_v = 152 kN. (4.2.5.2)
0.6 P_v = 0.6 × 636 = 382 kN
152 kN < 382 kN
Therefore, **the shear is low**.

For low shear, the moment capacity for a class 1 section is given by (4.2.5.2)
M_{cx} = $p_y S_x$
M_{cx} = 275 × 1470 × 10^{-3} = 404 kNm

Check limit to avoid irreversible deformation under serviceability loads. (4.2.5.1)
For a fixed-ended beam $M_{cx} \le 1.5\, p_y Z_x$
$1.5 p_y Z_x$ = 1.5 × 275 × 1300 × 10^{-3} = 536 kNm

Example 3 Unrestrained beam with end moments	Sheet 5 of 6	Rev

Therefore M_{cx} = 404 kNm
From above, M_x = 282 kNm
282 kNm < 404 kNm
Therefore, **the moment capacity is adequate.**

3.2.8 Lateral-torsional buckling

Basic requirements: $M_x \le M_b / m_{LT}$ 4.3.6.2
and $M_x \le M_{cx}$ (already checked in 3.2.7)

The buckling resistance moment M_b for a class 1 plastic section is given by 4.3.6.4
$M_b = p_b S_x$

where p_b is the bending strength and is dependent on the design strength p_y and the equivalent slenderness λ_{LT}.

$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$ 4.3.6.7

$$\lambda = \frac{L_E}{r_y} = \frac{6300}{41.2} = 153$$

$$\lambda/x = \frac{153}{37.9} = 4.0$$

For a section with equal flanges and $\lambda/x = 4.0$, Table 19
$v = 0.86$

For a class 1 plastic section, $\beta_W = 1.0$ 4.3.6.9
Therefore, $\lambda_{LT} = 0.872 \times 0.86 \times 153 \times 1.0 = 115$ Table 16
For $p_y = 275$ N/mm^2 and $\lambda_{LT} = 115$, Table 16 gives $p_b = 102$ N/mm^2
$M_b = 102 \times 1470 \times 10^{-3} = 150$ kNm
$M_b / m_{LT} = 150 / 0.46 = 326$ kNm
From above, M_x = 282 kNm
282 kNm < 326 kNm
Therefore, **the buckling resistance moment is adequate.**

3.2.9 Web bearing and buckling

The web bearing and buckling checks should be carried out at the supports and at the points of load application. However, as the reactions are transferred through end plates and the loads are applied through the bottom flange, there is no need to check the web for bearing and buckling in this example.

3.2.10 SLS deflection check

The deflections under unfactored imposed loads should be checked as in Example 2.

Adopt 457 x 191 x 67 UB in grade S275

Example 3 Unrestrained beam with end moments	Sheet 6 of 6	Rev

3.3 Blue Book approach

The member capacities and resistances calculated in section 3.2 could have been obtained directly from Volume 1[2].

Try 457 x 191 x 67 UB in grade S275

3.3.1 Shear Capacity

Vol 1
Page C-79

P_v = 636 kN
F_v = 152 kN
152 kN < 636 kN
Therefore, **the shear capacity is adequate.**

3.3.2 Moment capacity

Vol 1
Page C-59

M_{cx} = 405 kNm
M_x = 282 kNm
282 kNm < 405 kNm
Therefore, **the moment capacity is adequate.**

3.3.3 Buckling resistance moment

Vol 1
Page C-59

For L_E = 6.0 m, M_b = 159 kNm
For L_E = 7.0 m, M_b = 133 kNm
Interpolating, an approximate value for L_E = 6.3 m, is M_b = 151 kNm
M_b / m_{LT} = 151 / 0.46 = 328 kNm
M_x = 282 kNm
282 kNm < 328 kNm
Therefore, **the buckling resistance moment is adequate.**

Note:
m_{LT} values are not included in Volume 1 and must be obtained from Table 18 of BS 5950-1:2000.

Adopt 457 x 191 x 67 UB in grade S275

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CALCULATION SHEET

Job No. CDS 153		Sheet 1 of 5	Rev
Job Title Example no. 4			
Subject Simply supported beam with lateral restraint at load application points			
Client SCI	Made by MDH	Date Jun 2003	
	Checked by ASM	Date Oct 2003	

4 Simply supported beam with lateral restraint at load application points

4.1 Introduction

The beam shown in Figure 4.1 is laterally restrained at the ends and at the points of load application only. For the loading shown, design the beam in S275 steel.

Two solutions are presented: the full calculation and the simplified approach using Volume 1[2] (the "Blue Book" approach).

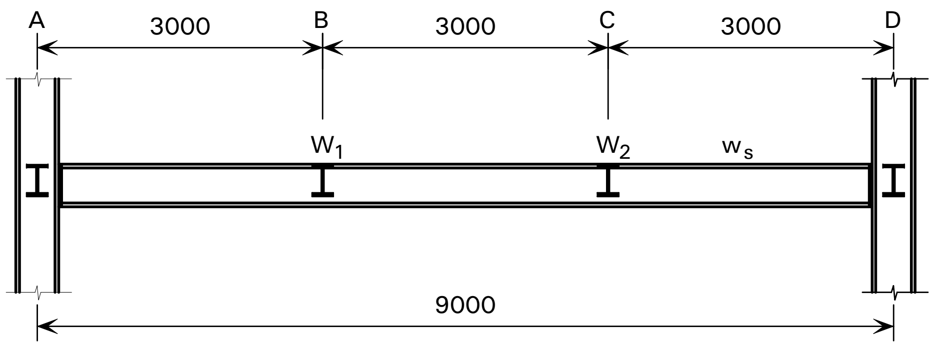

Figure 4.1

4.1.2 Loading (unfactored)

Dead loads:

Self-weight	w_s	= 3 kN/m
Point load	W_{1d}	= 40 kN
Point load	W_{2d}	= 20 kN

Imposed loads:

Point load	W_{1i}	= 60 kN
Point load	W_{2i}	= 30 kN

4.1.3 Load Factors

Table 2

Dead load factor	γ_{fd}	= 1.4
Imposed load factor	γ_{fi}	= 1.6

4.1.4 Factored loads

$w_s' = w_s\,\gamma_{fd} = (3 \times 1.4) = 4.2$ kN/m

$W_1' = W_{1d}\,\gamma_{fd} + W_{1i}\,\gamma_{fi} = (40 \times 1.4) + (60 \times 1.6) = 152$ kN

$W_2' = W_{2d}\,\gamma_{fd} + W_{2i}\,\gamma_{fi} = (20 \times 1.4) + (30 \times 1.6) = 76$ kN

The design shear forces and bending moments due to factored loads are as shown in Figure 4.2.

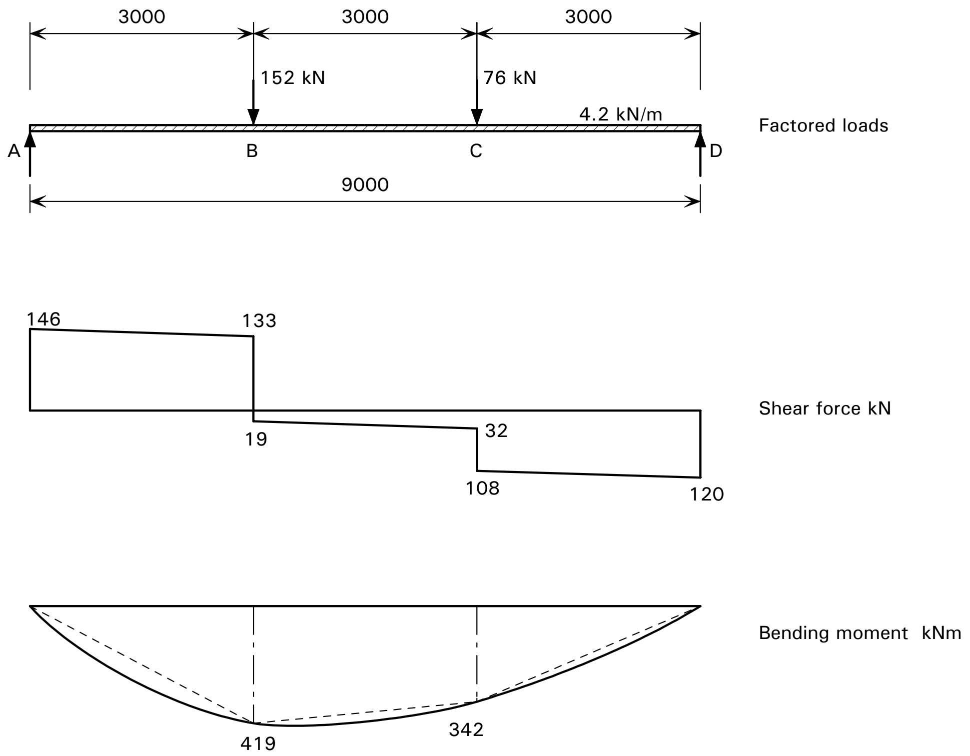

Figure 4.2

4.2 Member checks

4.2.1 Determine the effective length

Since the beam is restrained at the points of load application, there are three segments to consider. However, from the bending moment diagram (Figure 4.2), BC is the critical segment and is, therefore, the only segment considered in this example. 4.3.5

For segment length BC, the effective length $L_E = 1.0L_{LT} = 3000$ mm. 4.3.5.2

4.2.2 Calculate m_{LT}

Since there is no loading between the lateral restraints at B and C (apart from the self-weight of the beam, which is considered insignificant), m_{LT} can be obtained directly from Table 18 for a known value of β. 4.3.6.6

β is the ratio of the bending moments at points B and C,

i.e. $\beta = \dfrac{\text{moment at C}}{\text{moment at B}} = \dfrac{342}{419} = 0.82$

From Table 18, $m_{LT} = 0.93$ Table 18

4.2.3 Trial section

The initial trial section size is selected to give a suitable moment capacity and resistance to lateral-torsional buckling. This initial member size is then checked to ensure its suitability in all other respects.

For an unrestrained beam, the maximum major axis moment M_x must satisfy the following conditions:

$M_x \leq M_{cx}$ and $M_x \leq M_b / m_{LT}$ 4.3.6.2

The chosen beam must, therefore, satisfy:
$M_{cx} \geq 419$ kNm and
$M_b \geq 419 \times 0.93 = 390$ kNm for an effective length of 3.0 m.

Try 457 x 191 x 82 UB in grade S275

From member capacity tables: — Vol 1 Page C-59

Moment capacity	M_c	= 504 kNm
Buckling resistance	M_b	= 396 kNm for L_E = 3.0 m

From section property tables: — Vol 1 Page B-4

Depth	D	= 460.0 mm
Width	B	= 191.3 mm
Web thickness	t	= 9.9 mm
Flange thickness	T	= 16.0 mm
Depth between fillets	d	= 407.6 mm
Plastic modulus (Vol 1 Page B-5)	S_x	= 1830 cm^3
Elastic modulus	Z_x	= 1610 cm^3
Radius of gyration	r_y	= 4.23 cm
Buckling parameter	u	= 0.879
Torsional index	x	= 30.8

Local buckling ratios: — Vol 1 Page B-4

Flange	b/T	= 5.98
Web	d/t	= 41.2

4.2.4 Classify the cross section

Grade of steel = S275 — 3.1.1 Table 9
T = 16.0 mm
Therefore p_y = 275 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{275}} = 1.0$$

For the outstand element of a compression flange, the limiting b/T for a class 1 plastic flange is 9ε. Limiting $b/T = 9\varepsilon = 9.0$ — 3.5.2 Table 11
The actual $b/T = 5.98 < 9.0$
Therefore, **the flange is class 1**.

Since the section is symmetrical and subject to pure bending, the neutral axis is at mid depth. For this case, the limiting d/t for a class 1 plastic web is 80ε. — 3.5.2 Table 11
Limiting $d/t = 80\varepsilon = 80.0$
The actual $d/t = 41.2 < 80.0$
Therefore, **the web is class 1**.

The flange and the web are both class 1, therefore, **the cross section is class 1**.

4.2.5 Check for shear buckling

If the d/t ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000[1]. — 4.2.3
$70\varepsilon = 70.0$
In this case, $d/t = 41.2 < 70\varepsilon$, so **there is no need to check for shear buckling**.

Example 4 Beam with lateral restraint at load application points	Sheet 4 of 5	Rev

4.2.6 Check the shear capacity

Basic requirement $F_v \le P_v$ — 4.2.3

$P_v = 0.6\, p_y A_v$

$p_y = 275$ N/mm^2

$A_v = tD = 9.9 \times 460.0 = 4554$ mm^2

$P_v = 0.6 \times 275 \times 4554 \times 10^{-3} = 751$ kN

The highest shear force occurs at A, where $F_v = 146$ kN

146 kN < 751 kN

Therefore, **the shear capacity is adequate.**

4.2.7 Check the moment capacity

Basic requirement $M_x \le M_{cx}$ — 4.2.5.2

Check whether the shear is "high" (i.e. $F_v > 0.6\, P_v$) or "low" at the point of maximum moment. The maximum moment occurs at B, where $F_v = 133$ kN.

$0.6\, P_v = 0.6 \times 751 = 451$ kN

133 kN < 451 kN

Therefore, **the shear is low**.

For low shear, the moment capacity for a class 1 section is given by — 4.2.5.2

$M_{cx} = p_y\, S_x$

$M_{cx} = 275 \times 1830 \times 10^{-3} = 503$ kNm

Check limit to avoid irreversible deformation under serviceability loads. — 4.2.5.1

For a simply supported beam $M_{cx} \le 1.2\, p_y Z_x$

$1.2 p_y Z_x = 1.2 \times 275 \times 1610 \times 10^{-3} = 531$ kNm

Therefore $M_{cx} = 503$ kNm

From above, $M_x = 419$ kNm

419 kNm < 503 kNm

Therefore, **the moment capacity is adequate.**

4.2.8 Lateral-torsional buckling

Basic requirements: $M_x \le M_b / m_{LT}$ — 4.3.6.2

and $M_x \le M_{cx}$ (already checked in 4.2.7)

The buckling resistance moment M_b for a class 1 plastic section is given by — 4.3.6.4

$M_b = p_b S_x$

where p_b is the bending strength and is dependent on the design strength p_y and the equivalent slenderness λ_{LT}.

$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$ — 4.3.6.7

$$\lambda = \frac{L_E}{r_y} = \frac{3000}{42.3} = 70.9$$

$$\lambda/x = \frac{70.9}{30.8} = 2.3$$

For a section with equal flanges and $\lambda/x = 2.3$, — Table 19

$v = 0.94$

For a class 1 plastic section, $\beta_w = 1.0$ — 4.3.6.9

Example 4 Beam with lateral restraint at load application points	Sheet 5 of 5	Rev

Therefore, $\lambda_{LT} = 0.879 \times 0.94 \times 70.9 \times 1.0 = 58.6$

For $p_y = 275$ N/mm^2 and $\lambda_{LT} = 58.6$, Table 16 gives $p_b = 217$ N/mm^2 — Table 16

$M_b = 217 \times 1830 \times 10^{-3} = 397$ kNm

$M_b / m_{LT} = 397 / 0.93 = 427$ kNm

From above, $M_x = 419$ kNm

419 kNm < 427 kNm

Therefore, **the buckling resistance moment is adequate.**

4.2.9 Web bearing and buckling

There is no need to check the web for bearing and buckling in this example, because the secondary beams are connected into the webs of the primary beams and flexible end plates are used to connect the beams to the columns.

4.2.10 SLS deflection check

The deflections under unfactored imposed loads should be checked as in Example 2.

Adopt 457 x 191 x 82 UB in grade S275

4.3 Blue Book approach

The member capacities and resistances calculated in section 4.2 could have been obtained directly from Volume 1[2].

Try 457 x 191 x 82 UB in grade S275

4.3.1 Shear Capacity

Vol 1 Page C-79

$P_v = 751$ kN

$F_v = 146$ kN

146 kN < 751 kN

Therefore, **the shear capacity is adequate.**

4.3.2 Moment capacity

Vol 1 Page C-59

$M_{cx} = 504$ kNm

$M_x = 419$ kNm

419 kNm < 504 kNm

Therefore, **the moment capacity is adequate.**

4.3.3 Buckling resistance moment

Vol 1 Page C-59

For $L_E = 3.0$ m, $M_b = 396$ kNm

$M_b / m_{LT} = 396 / 0.93 = 426$ kNm

$M_x = 419$ kNm

419 kNm < 426 kNm

Therefore, **the buckling resistance moment is adequate.**

Note

m_{LT} values are not included in Volume 1 and must be obtained from Table 18 of BS 5950-1:2000.

Adopt 457 x 191 x 82 UB in grade S275

The Steel Construction Institute Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 623345 Fax: (01344) 622944 CALCULATION SHEET	Job No. CDS 153	Sheet 1 of 7	Rev
	Job Title Example no. 5		
	Subject Simply supported beam with lateral restraint at load application points using a class 3 UC		
	Client SCI	Made by MDH	Date Jun 2003
		Checked by ASM	Date Oct 2003

5 Simply supported beam with lateral restraint at load application points using a class 3 UC

5.1 Introduction

The beam shown in Figure 5.1 is laterally restrained at the ends and at the points of load application only. For the loading shown, design the beam in S355 steel using a Universal Column section.

Two solutions are presented: the full calculation and the simplified approach using Volume 1[2] (the "Blue Book" approach).

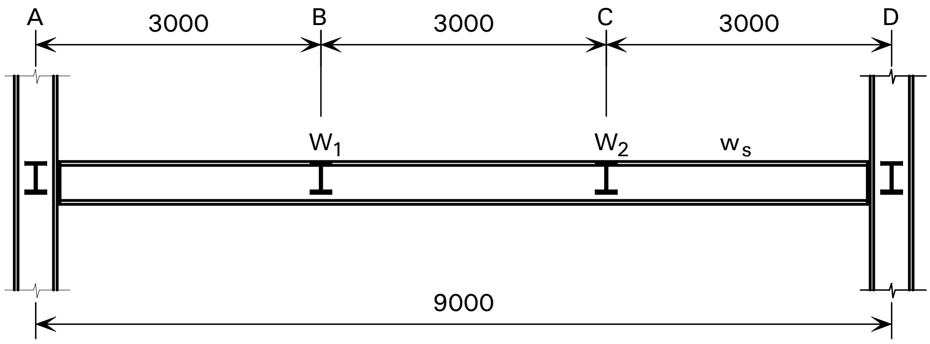

Figure 5.1

5.1.1 Factored loads

w_s' = 3.4 kN/m (self weight)
W_1' = 122 kN
W_2' = 61 kN

The shear forces and bending moments are as shown in Figure 5.2.

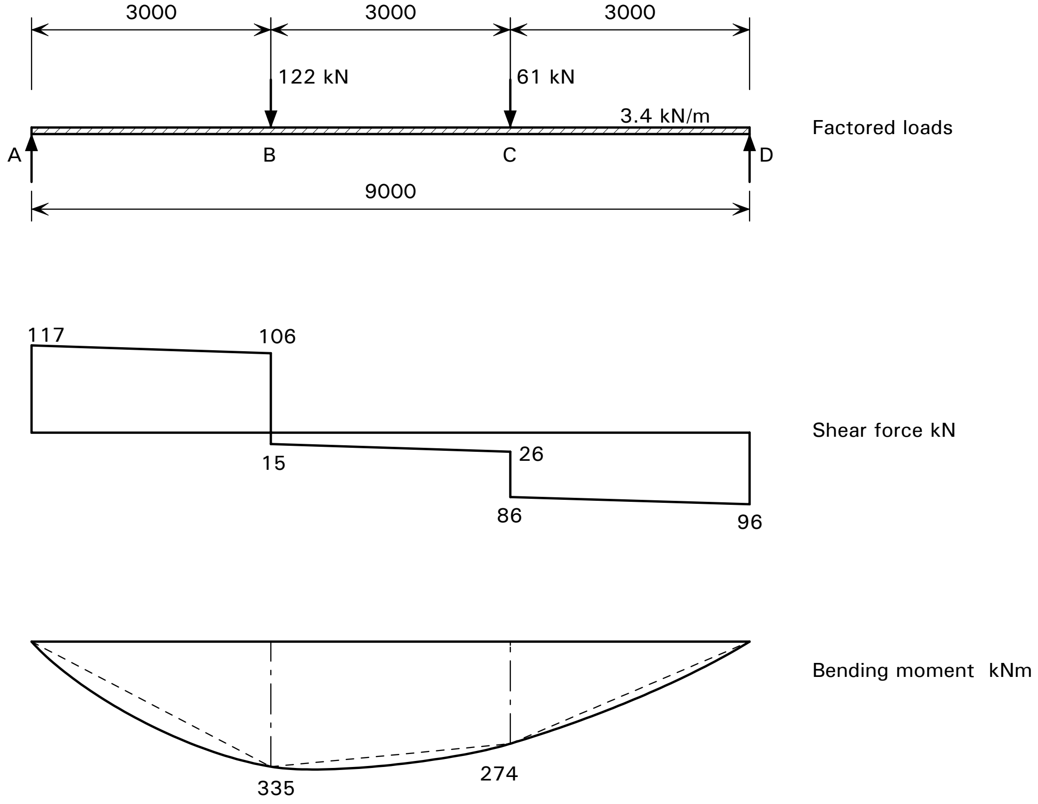

Figure 5.2

5.2 Member checks

5.2.1 Determine the effective length

Since the beam is restrained at the points of load application, there are three segments to consider. However, from the bending moment diagram (Figure 5.2), BC is the critical segment and is, therefore, the only segment considered in this example. 4.3.5

For segment length BC, the effective length $L_E = 1.0L_{LT} = 3000$ mm. 4.3.5.2

5.2.2 Calculate m_{LT}

Since there is no loading between the lateral restraints at B and C (apart from the self-weight of the beam, which is considered insignificant), m_{LT} can be obtained directly from Table 18 for a known value of β. 4.3.6.6

β is the ratio of the bending moments at points B and C,

i.e. $\beta = \dfrac{\text{moment at C}}{\text{moment at B}} = \dfrac{274}{335} = 0.82$

From Table 18, $m_{LT} = 0.93$ Table 18

5.2.3 Trial section

The initial trial section size is selected to give a suitable moment capacity and resistance to lateral-torsional buckling. This initial member size is then checked to ensure its suitability in all other respects.

Example 5 Beam with restraint at load points using class 3 UC	Sheet 3 of 7	Rev

For an unrestrained beam, the maximum major axis moment M_x must satisfy the following conditions: — 4.3.6.2

$M_x \le M_{cx}$ and $M_x \le M_b / m_{LT}$

The chosen beam must, therefore, satisfy:

$M_{cx} \ge 335$ kNm and

$M_b \ge 335 \times 0.93 = 312$ kNm for an effective length of 3.0 m.

Try 254 x 254 x 73 UC in grade S355

From member capacity tables: — Vol 1 Page D-61

Moment capacity $M_c = 350$ kNm

Buckling resistance $M_b = 332$ kNm for $L_E = 3.0$ m

From section property tables:

Depth	D	= 254.1 mm	Vol 1
Width	B	= 254.6 mm	Page B-8
Web thickness	t	= 8.6 mm	
Flange thickness	T	= 14.2 mm	
Depth between fillets	d	= 200.3 mm	Vol 1
Plastic modulus	S_x	= 992 cm^3	Page B-9
Elastic modulus	Z_x	= 898 cm^3	
Radius of gyration	r_y	= 6.48 cm	
Buckling parameter	u	= 0.849	
Torsional index	x	= 17.3	

Local buckling ratios:

Flange	b/T	= 8.96	Vol 1
Web	d/t	= 23.3	Page B-8

5.2.4 Classify the cross section

Grade of steel = S355 — 3.1.1 Table 9

$T < 16$ mm

Therefore p_y = 355 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{355}} = 0.88$$

For the outstand element of a compression flange, the limiting b/T for a class 1 plastic flange is 9ε. — 3.5.2 Table 11

Limiting b/T = 9ε = 9×0.88 = 7.92

The actual b/T = 8.96 > 7.92

Therefore, **the flange is not class 1.**

The limiting b/T for a class 2 compact flange is 10ε. — 3.5.2 Table 11

Limiting b/T = 10ε = 10×0.88 = 8.8

The actual b/T = 8.96 > 8.8

Therefore, **the flange is not class 2.**

The limiting b/T for a class 3 compact flange is 15ε. — 3.5.2 Table 11

Limiting b/T = 15ε = 15×0.88 = 13.2

The actual b/T = 8.96 < 13.2

Therefore, **the flange is class 3.**

Example 5 Beam with restraint at load points using class 3 UC	Sheet 4 of 7	Rev

Since the section is symmetrical and subject to pure bending, the neutral axis is at mid depth. For this case, the limiting d/t for a class 1 plastic web is 80ε. 3.5.2 Table 11

Limiting d/t $= 80\varepsilon = 80 \times 0.88 = 70.4$

The actual d/t $= 23.3 < 70.4$

Therefore, the **web is class 1**.

The flange is critical and, therefore, **the cross section is class 3.**

5.2.5 Check for shear buckling

If the d/t ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000[1]. 4.2.3

$70\varepsilon = 70 \times 0.88 = 61.6$

In this case, $d/t = 23.3 < 70\varepsilon$, so **there is no need to check for shear buckling**.

5.2.6 Check the shear capacity

Basic requirement $F_v \leq P_v$ 4.2.3

$P_v = 0.6\, p_y A_v$

$p_y = 355$ N/mm^2

$A_v = tD = 8.6 \times 254.1 = 2185$ mm^2

$P_v = 0.6 \times 355 \times 2185 \times 10^{-3} = 465$ kN

The highest shear force occurs at A, where $F_v = 117$ kN

117 kN < 465 kN

Therefore, the shear capacity is adequate.

5.2.7 Check the moment capacity

Basic requirement $M_x \leq M_{cx}$

Check whether the shear is "high" (i.e. $F_v > 0.6\, P_v$) or "low" at the point of maximum moment. The maximum moment occurs at B, where $F_v = 106$ kN. 4.2.5.2

$0.6\, P_v = 0.6 \times 465 = 279$ kN

106 kN < 279 kN

Therefore, **the shear is low**.

For low shear, the moment capacity for a class 3 section is given by 4.2.5.2

Either $M_{cx} = p_y Z_x$

Or $M_{cx} = p_y S_{x,eff}$

where $S_{x,eff}$ is the effective plastic modulus for a class 3 cross-section. $S_{x,eff}$ will be greater than the elastic modulus Z_x, but less than the plastic modulus S_x.

The first equation above is the traditional formula for a class 3 semi-compact section. It is simple to use, but limits the moment capacity of the section to the moment of first yield. By contrast, the second equation gives a more realistic approximation to the actual moment capacity of the section, but requires a little extra computational effort. Designers may choose either method. For the purpose of this example, the second equation will be used, since the section only just failed to be class 2 compact and will, therefore, have a moment capacity close to the fully plastic moment capacity.

Example 5 Beam with restraint at load points using class 3 UC	Sheet 5 of 7	Rev

The effective plastic modulus $S_{x,eff}$ is calculated as follows:

3.5.6.2

$$S_{x,eff} = Z_x + (S_x - Z_x)\left[\frac{\left(\frac{\beta_{3w}}{d/t}\right)^2 - 1}{\left(\frac{\beta_{3w}}{\beta_{2w}}\right)^2 - 1}\right]$$

3.5.6.2

$$\text{but } S_{x,eff} \le Z_x + (S_x - Z_x)\left[\frac{\left(\frac{\beta_{3f}}{b/T}\right)^2 - 1}{\left(\frac{\beta_{3f}}{\beta_{2f}}\right)^2 - 1}\right]$$

3.5.2
Table 11

$\beta_{2w} = 100\varepsilon = 100 \times 0.88 = 88$
$\beta_{3w} = 120\varepsilon = 120 \times 0.88 = 106$
$\beta_{2f} = 10\varepsilon = 10 \times 0.88 = 8.8$
$\beta_{3f} = 15\varepsilon = 15 \times 0.88 = 13.2$

3.5.6.2

$$S_{x,eff} = 898 + (992 - 898)\left[\frac{\left(\frac{106}{23.3}\right)^2 - 1}{\left(\frac{106}{88}\right)^2 - 1}\right] = 5004 \text{ cm}^3$$

3.5.6.2

$$\text{but } S_{x,eff} = 898 + (992 - 898)\left[\frac{\left(\frac{13.2}{8.96}\right)^2 - 1}{\left(\frac{13.2}{8.8}\right)^2 - 1}\right] = 986 \text{ cm}^3$$

In this case, the second equation is critical, since the flange is the critical element for this section under pure bending. The high value of $S_{x,eff}$ obtained from the first equation is meaningless, as it is greater than S_x, and should be ignored.

Therefore, $S_{x,eff} = 986 \text{ cm}^3$

4.2.5.2

$M_{cx} = 355 \times 986 \times 10^{-3} = 350$ kNm

(By comparison, $p_y Z_x = 319$ kNm)

4.2.5.1

Check limit to avoid irreversible deformation under serviceability loads.
For a simply supported beam $M_{cx} \le 1.2\, p_y Z_x$
$1.2 p_y Z_x = 1.2 \times 355 \times 898 \times 10^{-3} = 383$ kNm

Therefore $M_{cx} = 350$ kNm
From above, $M_x = 335$ kNm
335 kNm < 350 kNm
Therefore, **the moment capacity is adequate.**

Example 5 Beam with restraint at load points using class 3 UC	Sheet 6 of 7	Rev

5.2.8 Lateral-torsional buckling

Basic requirements: $M_x \leq M_b / m_{LT}$ — 4.3.6.2

and $M_x \leq M_{cx}$ (already checked in 5.2.7)

The buckling resistance moment M_b for a class 3 semi-compact section is given by

$M_b = p_b\, S_{x,eff}$ (or conservatively $M_b = p_b\, Z_x$) — 4.3.6.4

where p_b is the bending strength and is dependent on the design strength p_y and the equivalent slenderness λ_{LT}.

$$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$$ — 4.3.6.7

$$\lambda = \frac{L_E}{r_y} = \frac{3000}{64.8} = 46.3$$

$$\lambda/x = \frac{46.3}{17.3} = 2.7$$

For a section with equal flanges and $\lambda/x = 2.68$, — Table 19

$v = 0.92$

For a class 3 semi-compact section, β_W is given by — 4.3.6.9

$\beta_W = S_{x,eff} / S_{x,} = 986/992 = 0.99$

Therefore, $\lambda_{LT} = 0.849 \times 0.92 \times 46.3 \times (0.99)^{0.5} = 36$

For $p_y = 355$ N/mm^2 and $\lambda_{LT} = 36$, Table 16 gives $p_b = 338$ N/mm^2 — Table 16

$M_b = 338 \times 986 \times 10^{-3} = 333$ kNm

$M_b/m_{LT} = 333 / 0\ 93 = 358$ kNm

From above, $M_x = 335$ kNm

335 kNm < 358 kNm

Therefore, **the buckling resistance moment is adequate.**

5.2.9 Web bearing and buckling

There is no need to check the web for bearing and buckling in this example, because the secondary beams are connected into the webs of the primary beams and flexible end plates are used to connect the beams to the columns.

5.2.10 SLS deflection check

The deflections under unfactored imposed loads should be checked as in Example 2.

Adopt 254 x 254 x 73 UC in grade S355

Example 5 Beam with restraint at load points using class 3 UC	Sheet 7 of 7	Rev

5.3 Blue Book approach

The member capacities and resistances calculated in section 5.2 could have been obtained directly from Volume 1[2].

Try 254 x 254 x 73 UC in grade S355

5.3.1 Shear Capacity

P_v = 465 kN
F_v = 117 kN
117 kN < 465 kN
Therefore, **the shear capacity is adequate.**

Vol 1
Page D-81

5.3.2 Moment capacity

M_{cx} = 350 kNm
M_x = 335 kNm
335 kNm < 350 kNm
Therefore, **the moment capacity is adequate.**

Vol 1
Page D-61

5.3.3 Buckling resistance moment

For L_E = 3.0 m, M_b = 332 kNm
M_b / m_{LT} = 332 / 0.93 = 357 kNm
M_x = 335 kNm
335 kNm < 357 kNm
Therefore, **the buckling resistance moment is adequate.**

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Page D-61

Note

m_{LT} values are not included in Volume 1 and must be obtained from Table 18 of BS 5950-1:2000.

Adopt 254 x 254 x 73 UC in grade S355

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CALCULATION SHEET

Job No.	CDS 153	Sheet	1 of 7	Rev	
Job Title	Example no. 6				
Subject	Beam under combined bending and torsion using a UC section				
Client	SCI	Made by	MDH	Date	Jun 2003
		Checked by	ASM	Date	Oct 2003

6 Beam under combined bending and torsion using a UC section

6.1 Introduction

The beam shown in Figure 6.1 is unrestrained along its length. An eccentric load is applied to the bottom flange at the centre of the span in such a way that it does not provide any lateral restraint to the member. The end conditions are assumed to be simply supported for bending and fixed against torsion, but free for warping. For the factored loads shown, design the beam in S275 steel.

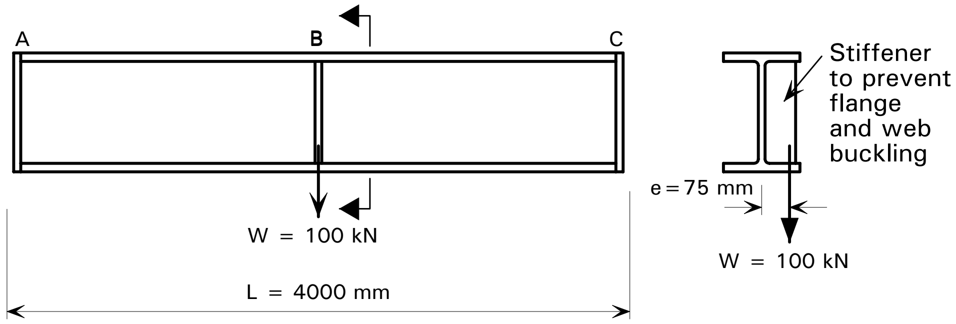


Figure 6.1

6.1.1 Loading

Replace the actual loading by an equivalent arrangement, comprising a vertical load applied through the shear centre and a torsional moment as shown below.

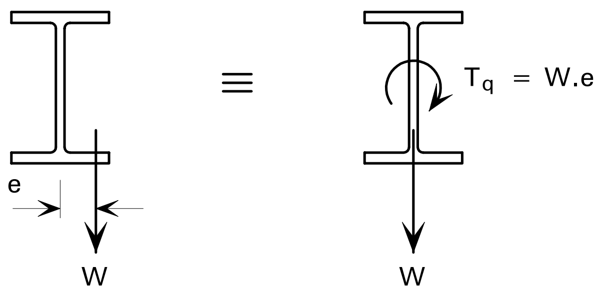


Figure 6.2

Example 6 Combined bending and torsion using a UC section	Sheet 2 of 7	Rev

Loadings due to plane bending and torsion are shown below.

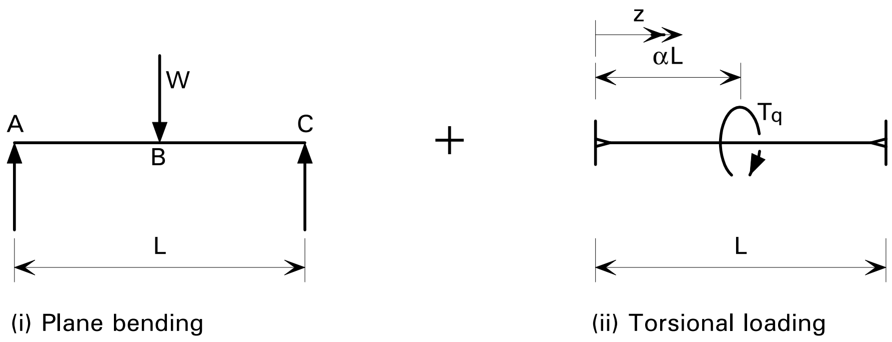


Figure 6.3

6.1.2 Factored loads

Distributed load (self-weight) $w = 1.0$ kN/m (assumed)
Point load $W = 100$ kN
Eccentricity $e = 75$ mm

6.1.3 Bending effects at ULS

Moment at B $M_{xB} = 102$ kNm
Shear at A $F_{vA} = 52$ kN
Shear at B $F_{vB} = 50$ kN

6.1.4 Torsional effects at ULS

Torsional moment $T_q = W \times e$
$T_q = 100 \times 75 \times 10^{-3} = 7.5$ kNm

6.2 Bending checks

4.3.5

6.2.1 Determine the effective length

Table 13

Since the beam is unrestrained between the supports, there is only one segment length to consider in this example, with a length equal to the beam length. In bending, the beam is simply supported.
Therefore, $L_E = 1.0L = 4000$ mm.

6.2.2 Calculate m_{LT}

4.3.6.6

If the self-weight is ignored (it is small compared to the point load), the bending moment diagram resembles the first of the four specific cases given in Table 18 of BS 5950-1:2000[1].

From Table 18, $m_{LT} = 0.85$ Table 18

Example 6 Combined bending and torsion using a UC section	Sheet 3 of 7	Rev

6.2.3 Trial section

In the previous examples, the initial trial section size was selected to give a suitable moment capacity and resistance to lateral-torsional buckling using Volume 1[2]. This initial member size was then checked to ensure its suitability in all other respects. In this example, due to the torsion, a beam chosen in this way is unlikely to be adequate. However, it is a useful starting point for the design procedure.

For an unrestrained beam, the maximum major axis moment M_x must satisfy the following conditions:

$M_x \leq M_{cx}$ and $M_x \leq M_b / m_{LT}$ — 4.3.6.2

The chosen beam must, therefore, satisfy:
$M_{cx} \geq 102$ kNm and
$M_b \geq 102 \times 0.85 = 86.7$ kNm for an effective length of 4.0 m.

To resist torsion, it is generally better to use a UC section rather than a UB. Referring to Volume 1[2], for bending only, a 203 × 203 × 46 UC in grade S275 is adequate, having a moment capacity M_c of 137 kNm and a buckling resistance moment M_b = 111 kNm for an effective length of 4.0 m. However, to allow for torsion, the next serial size up will be selected as the trial section. — Vol 1 Page C-61

Try 254 x 254 x 89 UC in grade S275

From member capacity tables: — Vol 1 Page C-61

Moment capacity	M_{cx}	= 324 kNm
Buckling resistance	M_b	= 299 kNm for L_E = 4.0 m

From section property tables: — Vol 1 Page B-8; Vol 1 Page B-9

Depth	D	= 260.3 mm
Width	B	= 256.3 mm
Web thickness	t	= 10.3 mm
Flange thickness	T	= 17.3 mm
Depth between fillets	d	= 200.3 mm
Second moment of area x axis	I_x	= 14300 cm^4
Radius of gyration y axis	r_y	= 6.55 cm
Plastic modulus x axis	S_x	= 1220 cm^3
Elastic modulus x axis	Z_x	= 1100 cm^3
Elastic modulus y axis	Z_y	= 379 cm^3
Torsion constant	J	= 102 cm^4

Local buckling ratios: — Vol 1 Page B-8

Flange	b/T	= 7.41
Web	d/t	= 19.4

6.2.4 Classify the cross section

Grade of steel = S275 — 3.1.1 Table 9
$T > 16.0$ mm
Therefore p_y = 265 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{265}} = 1.02$$

For the outstand element of a compression flange, the limiting b/T for a class 1 plastic flange is 9ε. — 3.5.2 Table 11
Limiting $b/T = 9\varepsilon = 9 \times 1.02 = 9.18$
The actual $b/T = 7.41 < 9.18$
Therefore, **the flange is class 1.**

Example 6 Combined bending and torsion using a UC section	Sheet 4 of 7	Rev

Since the section is symmetrical and subject to bending (without axial load), the neutral axis is at mid depth. For this case, the limiting *d/t* for a class 1 plastic web is 80ε. — 3.5.2 Table 11

Limiting *d/t* $= 80\varepsilon = 80 \times 1.02 = 81.6$
The actual *d/t* $= 19.4 < 81.6$
Therefore, the **web is class 1**.

The flange and the web are both class 1, therefore, **the cross section is class 1**.

6.2.5 Check for shear buckling

If the *d*/*t* ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000 [1]. — 4.2.3

$70\varepsilon = 70 \times 1.02 = 71.4$
In this case, $d/t = 19.4 < 70\varepsilon$, **so there is no need to check for shear buckling**.

6.2.6 Check the shear capacity

Basic requirement $F_v \leq P_v$ — 4.2.3

$P_v = 0.6\, p_y A_v$
$p_y = 265$ N/mm²
$A_v = tD = 10.3 \times 260.3 = 2681$ mm²
$P_v = 0.6 \times 265 \times 2681 \times 10^{-3} = 426$ kN
The highest shear force occurs at A, where $F_v = 52$ kN
52 kN < 426 kN
Therefore, **the shear capacity is adequate**.

6.2.7 Check the moment capacity

Basic requirement $M_x \leq M_{cx}$ — 4.2.5.2

Check whether the shear is "high" (i.e. $F_v > 0.6\, P_v$) or "low" at the point of maximum moment. The maximum moment occurs at B, where $F_v = 50$ kN.

$0.6\, P_v = 0.6 \times 426 = 256$ kN

50 kN < 256 kN
Therefore, **the shear is low**.

For low shear, the moment capacity for a class 1 section is given by: — 4.2.5.2

$M_{cx} = p_y\, S_x$
$M_{cx} = 265 \times 1220 \times 10^{-3} = 323$ kNm

Check limit to avoid irreversible deformation under serviceability loads. — 4.2.5.1

For a simply supported beam $M_{cx} \leq 1.2\, p_y Z_x$
$1.2 p_y Z_x = 1.2 \times 265 \times 1100 \times 10^{-3} = 350$ kNm
Therefore $M_{cx} = 323$ kNm
From above, $M_x = 102$ kNm
102 kNm < 323 kNm
Therefore, the moment capacity is adequate.

6.2.8 Lateral-torsional buckling

Basic requirements: $M_x \leq M_b / m_{LT}$ — 4.3.6.2
and $M_x \leq M_{cx}$ (already checked in 6.2.7)

The buckling resistance moment M_b for a class 1 plastic section is given by 4.3.6.4

$M_b \quad = p_b S_x$

where p_b is the bending strength and is dependent on the design strength p_y and the equivalent slenderness λ_{LT}.

4.3.6.7

$$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$$

$$\lambda = \frac{L_E}{r_y} = \frac{4000}{65.5} = 61.1$$

$$\lambda/x = \frac{61.1}{14.5} = 4.2$$

For a section with equal flanges and $\lambda/x = 4.21$, Table 19
$v = 0.85$

For a class 1 plastic section, $\beta_W = 1.0$ 4.3.6.9

Therefore, $\lambda_{LT} = 0.851 \times 0.85 \times 61.1 \times 1.0 = 44.2$ Table 16
For $p_y = 265$ N/mm^2 and $\lambda_{LT} = 44.2$, Table 16 gives $p_b = 244$ N/mm^2
$M_b = 244 \times 1220 \times 10^{-3} = 298$ kNm
$M_b / m_{LT} = 298 / 0.85 = 351$ kNm

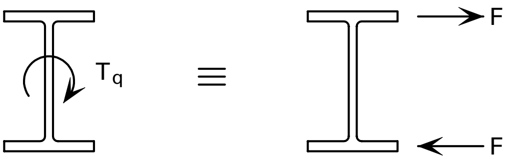
From above, $M_x = 102$ kNm
102 kNm < 351 kNm
Therefore, **the buckling resistance moment is adequate.**

6.3 Combined bending and torsion checks

A complete and very accurate method is given in P057 *Design of members subject to combined bending and torsion*[3]. However, the following simplified method may be used for I-sections subject to combined bending and torsion. The method has been used in practice in building design for many years. It ignores the pure torsion stiffness of the beam, but also ignores the small component of the major axis moment that is applied as a minor axis moment due to the twist of the section. The beam is then checked using the philosophy of BS 5950-1:2000[1] clause 4.9, which requires checks to clause 4.8.3 with the axial load taken as zero.

6.3.1 Equivalent flange force

The applied torque may be replaced by two equal and opposite lateral forces in the flanges as shown below.



The force F, acting in each flange, is given by:

$$F = \frac{T_q}{D - T} = \frac{7.5}{(260.3 - 17.3) \times 10^{-3}} = 30.9 \text{ kN}$$

Example 6 Combined bending and torsion using a UC section	Sheet 6 of 7	Rev

6.3.2 Lateral bending of the flange

4.8.3.2(a)

Basic requirement: $M_f \le M_{cf}$

This flange force is applied at the mid-span of the beam (at the same location as the applied torque). Since the flanges are free to rotate on plan at the supports, the maximum bending moment in the flange due to the lateral flange force is given by

$$M_f = \frac{FL}{4} = \frac{30.9 \times 4.0}{4} = 30.9 \text{ kNm}$$

The moment capacity of a class 1 plastic flange is given by

$M_{cf} = p_y S_f$

where S_f is the plastic modulus of the flange about its major axis (i.e. the minor axis of the beam).

$$S_f = \frac{TB^2}{4} = \frac{17.3 \times 256.3^2}{4} \times 10^{-3} = 284 \text{ cm}^3$$

$M_{cf} = 265 \times 284 \times 10^{-3} = 75.3$ kNm

30.9 kNm < 75.3 kNm

Therefore, **the moment capacity of the flange is adequate.**

6.3.3 Cross section check

4.8.3.3.1

Basic requirement: $\dfrac{M_x}{M_{cx}} + \dfrac{M_f}{M_{cf}} \le 1$

$$\frac{102}{323} + \frac{30.9}{75.3} = 0.316 + 0.410 = 0.73 < 1$$

Therefore, **the cross section check is OK.**

6.3.4 Buckling check

4.8.3.3.4
Table 26

Basic requirement: $\dfrac{m_{LT} M_{LT}}{M_b} + \dfrac{m_y M_f}{p_y Z_f} \le 1$

For the flange, $I_f = \dfrac{TB^3}{12} = \dfrac{17.3 \times (256.3)^3}{12} \times 10^{-4} = 2427 \text{ cm}^4$

$$Z_f = \frac{I_f}{B/2} = \frac{2427}{12.8} = 189.4 \text{ cm}^3$$

m_y is determined according to the shape of the bending moment diagram for the flange between restraints on the y axis. For the case of a simply supported beam with a centrally applied load, $m_y = 0.90$.

$$\frac{0.85 \times 102}{298} + \frac{0.9 \times 30.9 \times 10^6}{265 \times 189.4 \times 10^3} = 0.291 + 0.554 = 0.85 < 1$$

Therefore, **the buckling check is OK.**

6.4 Deflections and twist

It is common for deflection to govern the design of beams subject to torsion. The vertical deflections are calculated as in Example 2 using the SLS loads.

The twist can be calculated from the horizontal deflections of the two flanges.

For simplicity, the SLS loading may be calculated as:

$F_{SLS} = F_{ULS}/1.5$

$F_{ULS} = 30.9$ kN (See Section 6.3.1)

$F_{SLS} = 30.9/1.5 = 20.6$ kN

The horizontal deflection of each flange is given by:

$$\delta = \frac{F_{SLS} L^3}{48 EI_f} = \frac{20.6 \times 10^3 \times 4000^3}{48 \times 205000 \times 2427 \times 10^4} = 5.52 \text{ mm}$$

The maximum twist is given by:

$$\phi = \frac{2\delta}{D - T} = \frac{2 \times 5.52}{260.3 - 17.3} = 0.0454 \text{ radians} = 2.6 \text{ degrees}$$

This twist exceeds the recommended limit of 2 degrees in P057 *Design of members subject to combined bending and torsion*[3]. In practice, a heavier section would have to be used.

Note that this twist is in addition to any rotations due to movement of the connections or deflections of the supporting structure.

The Steel Construction Institute	Job No. CDS 153	Sheet 1 of 5	Rev
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 623345 Fax: (01344) 622944	Job Title Example no. 7		
	Subject RHS beam under combined bending and torsion		
CALCULATION SHEET	Client SCI	Made by MDH	Date Jun 2003
		Checked by ASM	Date Oct 2003

7 RHS beam under combined bending and torsion

7.1 Introduction

Redesign the member shown in Example 6 using a hot-finished rectangular hollow section (RHS).

7.1.1 Factored loads

Distributed load (self-weight) w = 1.0 kN/m (assumed)
Point load W = 100 kN
Eccentricity e = 75 mm

7.1.2 Bending effects at ULS

Moment at B M_{xB} = 102 kNm
Shear at A F_{vA} = 52 kN
Shear at B F_{vB} = 50 kN

7.1.3 Torsional effects at ULS

Torsional moment $T_q = W \times e$
$T_q = 100 \times 75 \times 10^{-3} = 7.5$ kNm

7.2 Bending checks

7.2.1 Determine the effective length

4.3.5

Since the beam is unrestrained between the supports, there is only one segment length to consider in this example, with a length equal to the beam length. In bending the beam is simply supported.
Therefore, $L_E = 1.0L = 4000$ mm.

Table 13

7.2.2 Trial section

Try 250 x 150 x 8 RHS in S275 (Hot finished)
From section property tables:

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Page B-24

Depth	D	= 250 mm
Width	B	= 150 mm
Web/flange thickness	t	= 8 mm
Area of cross section	A	= 60.8 cm^2
Second moment of area x axis	I_x	= 5110 cm^4
Radius of gyration y axis	r_y	= 6.15 cm
Plastic modulus x axis	S_x	= 501 cm^3
Elastic modulus x axis	Z_x	= 409 cm^3
Elastic modulus y axis	Z_y	= 306 cm^3
Torsion constant	J	= 5020 cm^4

Example 7 RHS beam under combined bending and torsion	Sheet 2 of 5	Rev

Torsional modulus constant $C = 506 \text{ cm}^3$

Local buckling ratios: (Vol 1 Page B-24)

Flange $b/t = 15.8$

Web $d/t = 28.3$

7.2.3 Classify the cross section

Grade of steel = S275 (3.1.1 Table 9)

$t < 16.0$ mm

Therefore $p_y = 275 \text{ N/mm}^2$

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{275}} = 1.0$$

For a hot finished RHS, the limiting b/t for a class 1 plastic flange is 28ε. (3.5.2 Table 12)

Limiting $b/t = 28\varepsilon = 28 \times 1.0 = 28$

The actual $b/t = 15.8 < 28$

Therefore, **the flange is class 1**.

Since the section is symmetrical and subject to bending (i.e. no axial load), the neutral axis is at mid depth. For this case, the limiting d/t for a class 1 plastic web is 64ε. (3.5.2 Table 12)

Limiting $d/t = 64\varepsilon = 64 \times 1.0 = 64$

The actual $d/t = 28.3 < 64$

Therefore, **the web is class 1**.

The flange and the web are both class 1, therefore, **the cross section is class 1**.

7.2.4 Check for shear buckling

If the d/t ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000[1]. (4.2.3)

$70\varepsilon = 70 \times 1.0 = 70$

In this case, $d/t = 28.3 < 70\varepsilon$, so **there is no need to check for shear buckling**.

7.2.5 Check the shear capacity

Basic requirement $F_v \leq P_v$ (4.2.3)

$P_v = 0.6\, p_y A_v$

$p_y = 275 \text{ N/mm}^2$

$A_v = AD/(D+B) = 6080 \times 250/(250+150) = 3800 \text{ mm}^2$

$P_v = 0.6 \times 275 \times 3800 \times 10^{-3} = 627 \text{ kN}$

The highest shear force occurs at A, where $F_v = 52$ kN

52 kN < 627 kN

Therefore, **the shear capacity is adequate.**

7.2.6 Check the moment capacity

Basic requirement $M_x \leq M_{cx}$ (4.2.5)

Without calculation, it is clear that the shear is "low" in this case.

For low shear, the moment capacity for a class 1 section is given by: (4.2.5.2)

$M_{cx} = p_y S_x$

$M_{cx} = 275 \times 501 \times 10^{-3} = 138 \text{ kNm}$

Check limit to avoid irreversible deformation under serviceability loads. (4.2.5.1)

For a simply supported beam $M_{cx} \leq 1.2\, p_y Z_x$

Example 7 RHS beam under combined bending and torsion	Sheet 3 of 5	Rev

$1.2p_y Z_x = 1.2 \times 275 \times 409 \times 10^{-3} = 135$ kNm

Therefore $M_{cx} = 135$ kNm

From above, $M_x = 102$ kNm

102 kNm < 135 kNm

Therefore, **the moment capacity is adequate.**

7.2.7 Lateral-torsional buckling

For an RHS, there is no need to consider lateral-torsional buckling, unless the slenderness L_E/r_y exceeds the value given in Table 15 (dependent on D/B). 4.3.6.1 Table 15

$D/B = 250/150 = 1.67$

For $D/B = 1.67$, the limiting L_E/r_y from Table 15 is 435.

The actual $L_E/r_y = 4000/61.5 = 65$

Therefore, there is no need to consider lateral-torsional buckling in this example.

$M_b = M_{cx} = 135$ kN.

7.3 Combined bending and torsion checks

As a conservative alternative to the method proposed in P057[3], the following simplified method may be used for RHS subject to combined bending and torsion.

7.3.1 Internal torque

The internal torque, T_o, within the member due to the applied torque T_q is obtained by considering the torsional equilibrium of the member as shown in Figure 7.1.

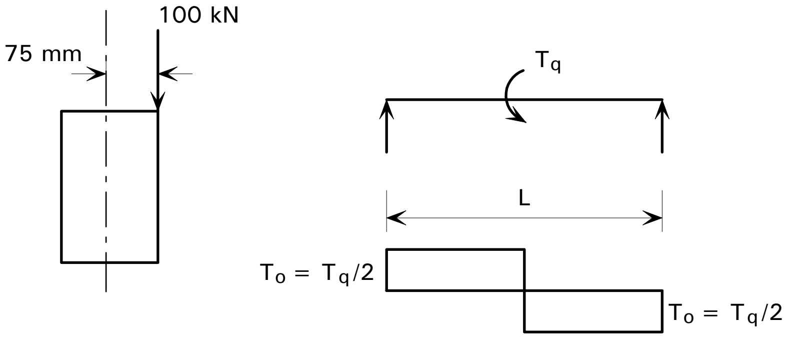


Figure 7.1

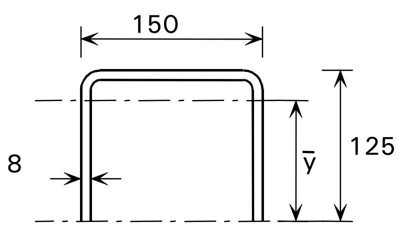
$$T_o = \frac{T_q}{2} = \frac{7.5}{2} = 3.75 \text{ kNm.}$$

7.3.2 Stresses due to bending

Normal stress in the outer fibre of the flange:

$$\sigma_{bx} = \frac{M_x}{Z_x} = \frac{102 \times 10^6}{409 \times 10^3} = 249 \text{ N/mm}^2$$

Shear stress due to bending:



$$\tau_b = \frac{F_{vA} Q_w}{I_x t_1}$$

$$Q_w = A_1 \bar{y}$$

Example 7 RHS beam under combined bending and torsion	Sheet 4 of 5	Rev

$$A_1 = \frac{A}{2} = \frac{6080}{2} = 3040 \text{ mm}^2$$

$$\bar{y} = \frac{(125^2 \times 8) + (134 \times 8 \times 121)}{A_1} = 83.8 \text{ mm}$$

$$Q_w = 3040 \times 83.8 \times 10^{-3} = 254.8 \text{ cm}^3$$

$$\tau_b = \frac{52 \times 10^3 \times 254.8 \times 10^3}{5110 \times 10^4 \times (2 \times 8)} = 16.2 \text{ N/mm}^2$$

7.3.3 Shear stresses due to torsion

$$\tau_t = \frac{T_o}{C} = \frac{3.75 \times 10^6}{506 \times 10^3} = 7.4 \text{ N/mm}^2$$

7.3.4 Total shear stress

$$\tau = \tau_b + \tau_t = 16.2 + 7.4 = 23.6 \text{ N/mm}^2$$

7.3.5 Von Mises interaction check

Basic requirement: $\sqrt{\sigma_{bx}^2 + 3\tau^2} \leq p_y$

$$\sqrt{249^2 + 3 \times 23.6^2} = 252 \text{ N/mm}^2 < 275 \text{ N/mm}^2$$

Note that this is very slightly conservative because it uses the maximum shear stress due to the shear force, which is not coexistent with the maximum bending stresses. The shear stresses from the shear force could be calculated with sufficient accuracy by using:

$$\tau_b = \frac{F_v}{A_v}$$

where F_v and A_v are as defined in BS 5950-1:2000[1] clause 4.2.3.

Example 7 RHS beam under combined bending and torsion	Sheet 5 of 5	Rev

7.4 Deflections

It is common for deflection to govern the design of beams subject to torsion.

The twist along the member is given by:

$$\phi = \int \frac{d\phi}{dz} dz$$

$$\frac{d\phi}{dz} = \frac{T}{GJ}$$

For simplicity, the SLS torque may be calculated as:

$T_{SLS} = T_{ULS}/1.5$

$T_{ULS} = T_o = 3.75$ kNm (See Section 7.3.1)

$T_{SLS} = 3.75/1.5$ 2.5 kNm

$$G = \frac{E}{2(1+v)} \approx 79{,}000 \text{ N/mm}^2$$

$$\frac{d\phi}{dz} = \frac{2.5 \times 10^6}{79 \times 10^3 \times 5020 \times 10^4} = 6.3 \times 10^{-7} \text{ radians/mm}$$

Twist $\phi = \frac{d\phi}{dz} \times \frac{L}{2} = 6.3 \times 10^{-7} \times 2000 = 1.26 \times 10^{-3}$ radians $\approx$ 0.07 degrees

The large torsional stiffness of hollow sections results in small twists along the member. However there may be additional twist from the connections and the supporting structure.

<u>**Adopt 250 x 150 x 8 RHS in S275 (Hot finished)**</u>

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CALCULATION SHEET

Job No. CDS 153		Sheet 1 of 9	Rev
Job Title Example no. 8			
Subject Continuous beam designed elastically			
Client SCI	Made by MDH	Date Jun 2003	
	Checked by ASM	Date Oct 2003	

8 Continuous beam designed elastically

8.1 Introduction

The continuous non-composite beam shown in Figure 8.1 has its top flange fully restrained laterally by a composite slab supported on secondary beams. The bottom flange is restrained at the supports and at the points of load application by the secondary beams. The dead load is 50 kN/m and the imposed load is 75 kN/m from point 1 to point 6 and 100 kN/m from point 6 to point 8. Design the beam elastically in S275 steel using a uniform section throughout.

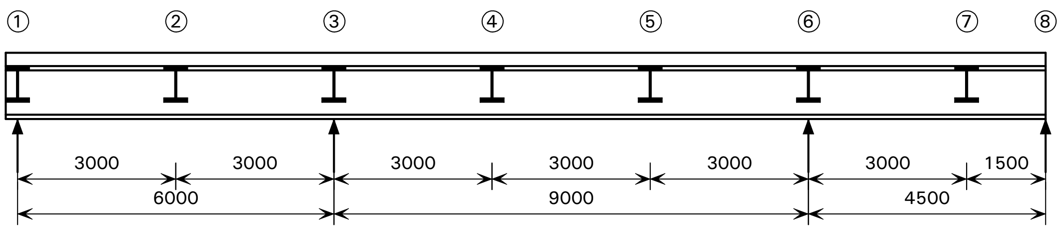


Figure 8.1

8.1.2 Loading

Taking the self weight of the beam as concentrated at the secondary beams, the bending moments and shear forces in the beam are related to the point loads at points 2, 4, 5 and 7 as shown in Figure 8.2.

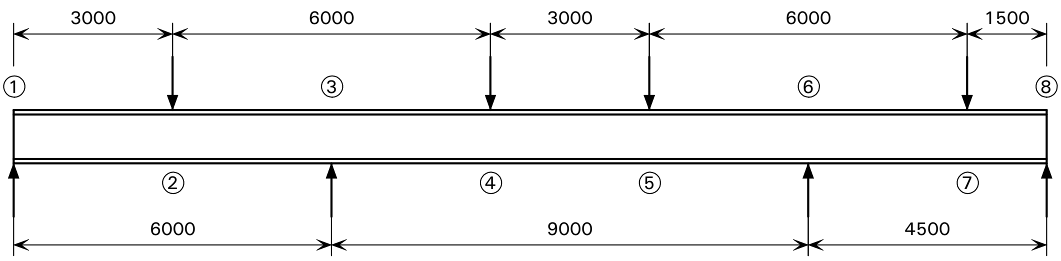


Figure 8.2

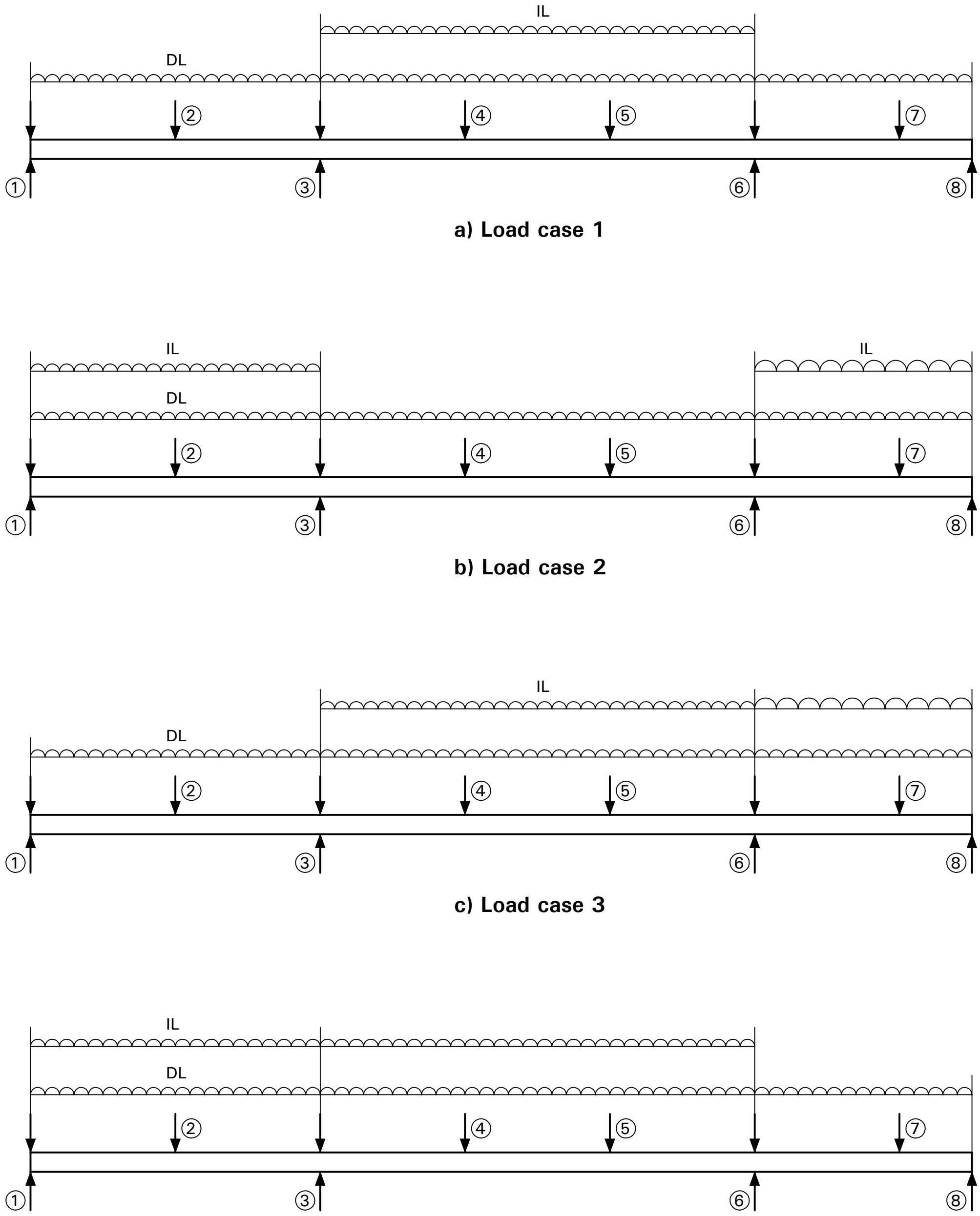
	Unfactored loads (kN)		Factored loads (kN)	
Location	**Dead load**	**Imposed load**	**Dead load**	**Imposed load**
2	150	225	210	360
4	150	225	210	360
5	150	225	210	360
7	150	300	210	480

8.1.3 Load cases

Vertical loads should be arranged in the most unfavourable but realistic pattern for each element. The four possible load cases are shown below in Figure 8.3, with the corresponding bending moments and shear forces in Figure 8.4.

Maximum moment is at point 3 for load case 4.
Design moment $M_x = M_{x\,@\,3} = 1029$ kNm

Maximum shear is at point 6 for load case 3.
Design shear force $F_v = F_{v\,@\,6} = 593$ kN



a) Load case 1

b) Load case 2

c) Load case 3

d) Load case 4

Figure 8.3 *Load cases*

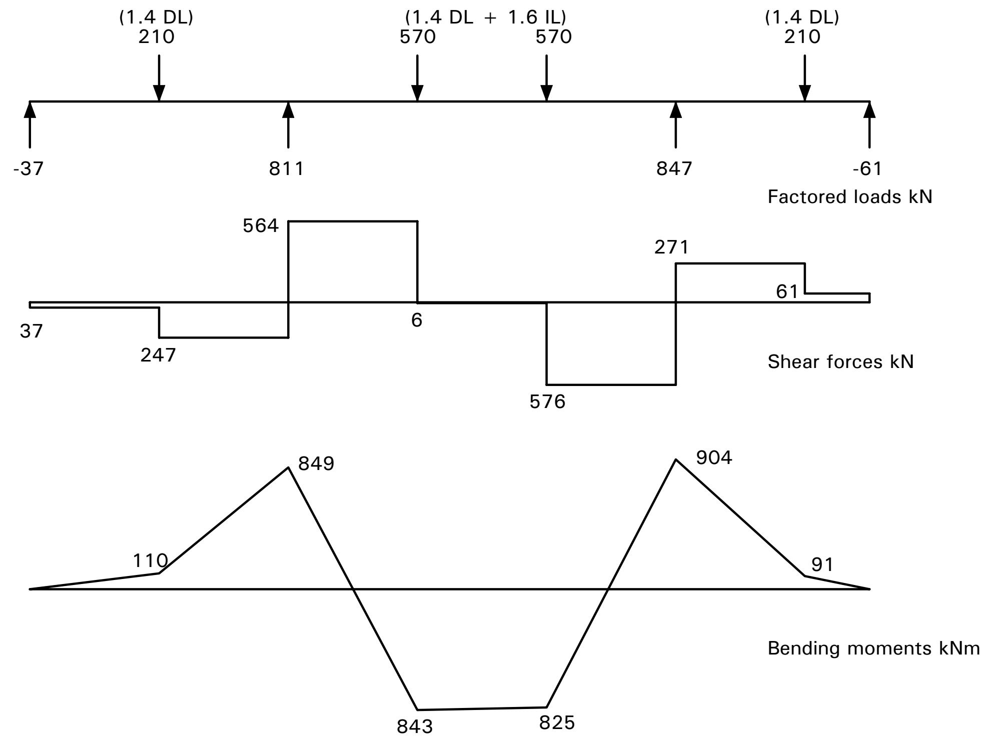


Figure 8.4(a) *Load case 1*

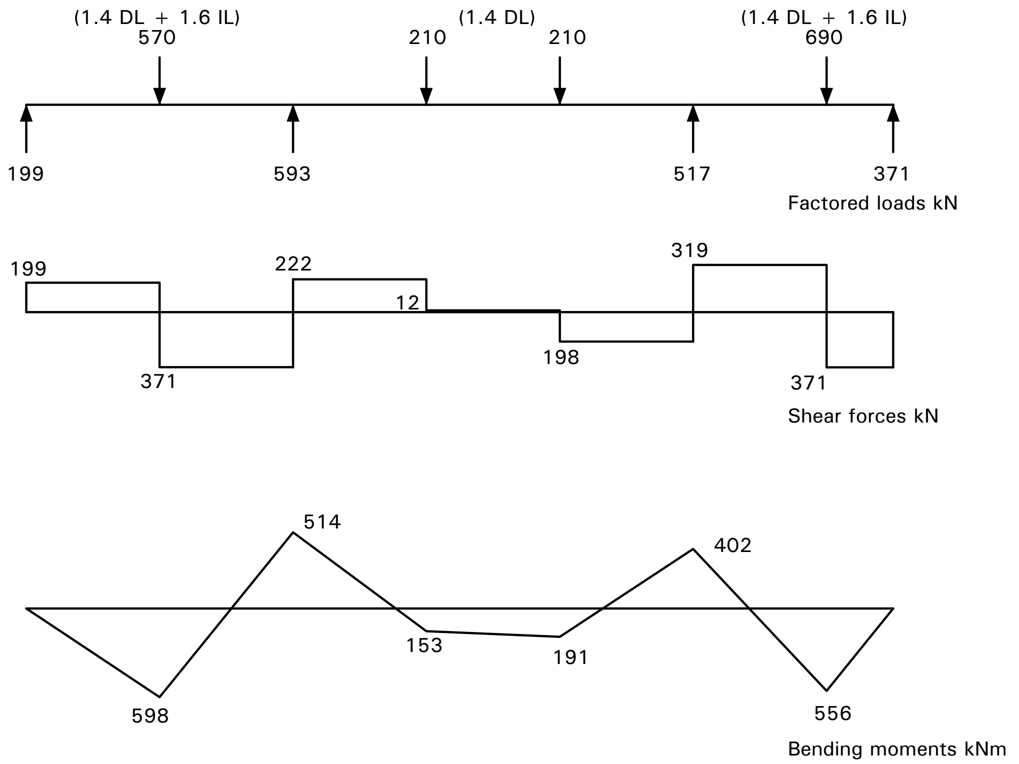


Figure 8.4(b) *Load case 2*

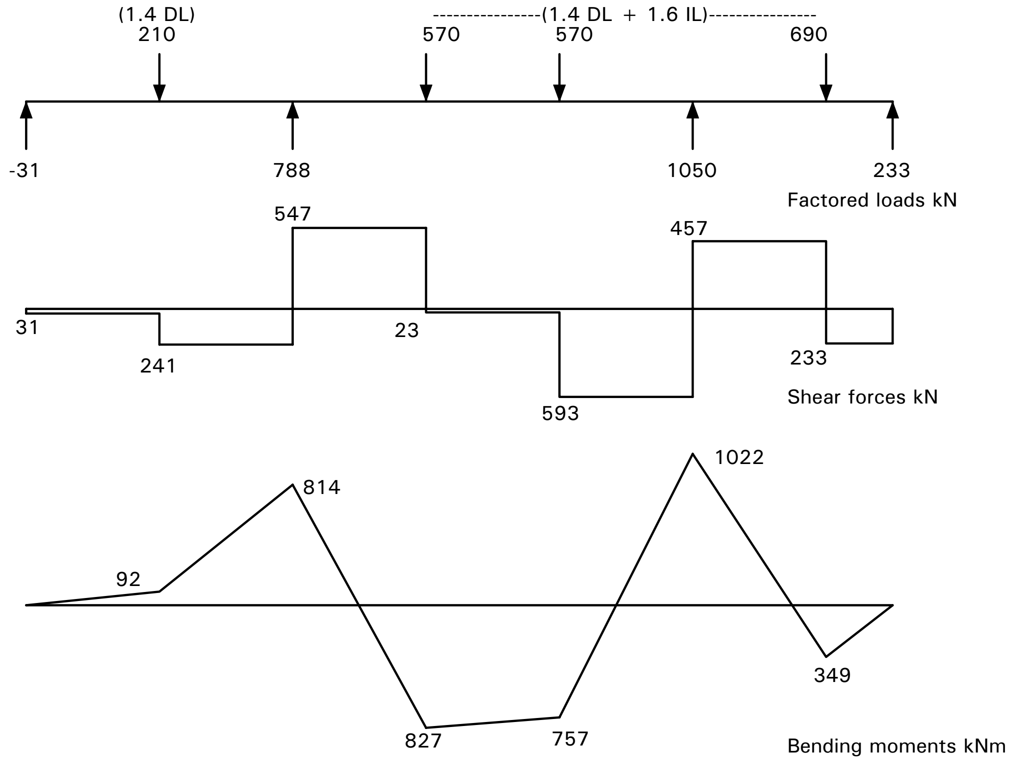


Figure 8.4(c) *Load case 3*

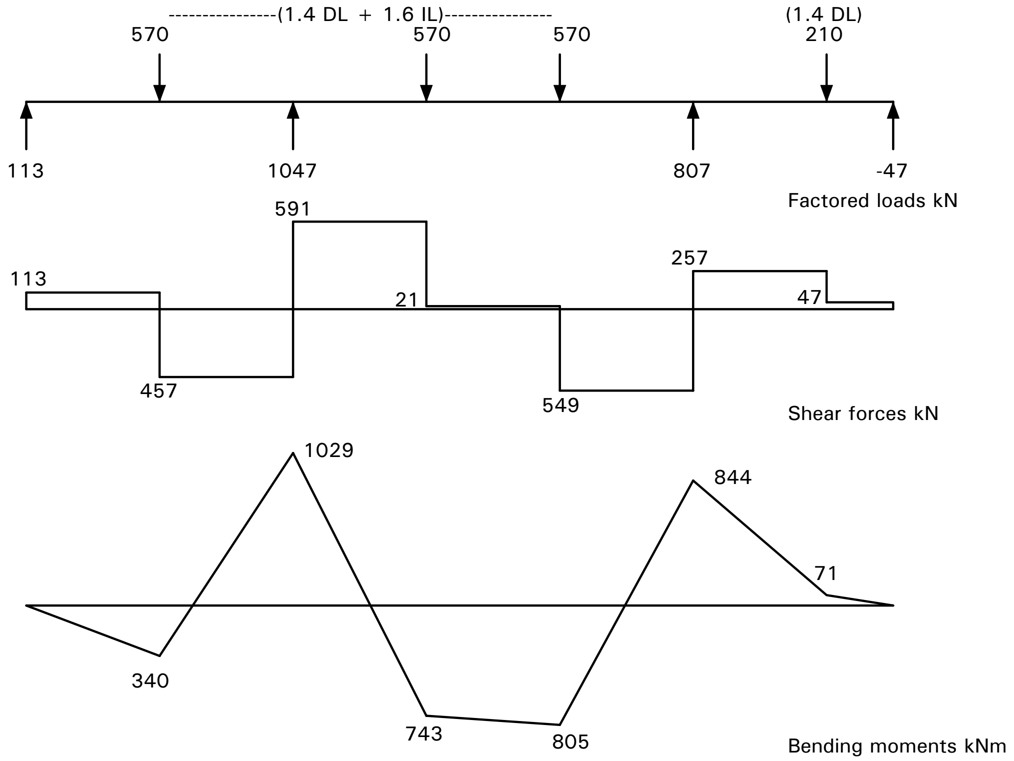


Figure 8.4(d) *Load case 4*

Example 8 Continuous beam designed elastically	Sheet 5 of 9	Rev

8.2 Member design

8.2.1 Trial section

Try 686 x 254 x 125 UB in grade S275

From section property tables: — Vol 1 Page B-2

Depth	D	= 677.9 mm
Width	B	= 253.0 mm
Web thickness	t	= 11.7 mm
Flange thickness	T	= 16.2 mm
Depth between fillets	d	= 615.1 mm
Radius of gyration	r_y	= 5.24 cm
Plastic modulus	S_x	= 3990 cm^3
Elastic modulus	Z_x	= 3480 cm^3
Buckling parameter	u	= 0.863
Torsional index	x	= 43.8

Vol 1 Page B-3

Local buckling ratios:

Flange	b/T	= 7.81
Web	d/t	= 52.6

Vol 1 Page B-2

8.2.2 Classify the cross section

Grade of steel = S275 — 3.1.1 Table 9

$T > 16.0$ mm

Therefore $p_y = 265$ N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{265}} = 1.02$$

For the outstand element of a compression flange, the limiting b/T for a class 1 plastic flange is 9ε. — 3.5.2 Table 11

Limiting $b/T = 9\varepsilon = 9 \times 1.02 = 9.18$

The actual $b/T = 7.81 < 9.18$

Therefore, **the flange is class 1**.

Since the section is symmetrical and subject to pure bending, the neutral axis is at mid depth. For this case, the limiting d/t for a class 1 plastic web is 80ε. — 3.5.2 Table 11

Limiting $d/t = 80\varepsilon = 80 \times 1.02 = 81.6$

The actual $d/t = 52.6 < 81.6$

Therefore, the **web is class 1**.

The flange and the web are both class 1, therefore, **the cross section is class 1**.

8.2.3 Check for shear buckling

If the d/t ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000[1]. — 4.2.3

$70\varepsilon = 70 \times 1.02 = 71.4$

In this case, $d/t = 52.6 < 70\varepsilon$, so **there is no need to check for shear buckling**.

8.2.4 Check the shear capacity

Basic requirement $F_v \leq P_v$ — 4.2.3

$P_v = 0.6\, p_y A_v$

$p_y = 265$ N/mm^2

$A_v = tD = 11.7 \times 677.9 = 7931$ mm^2

$P_v = 0.6 \times 265 \times 7931 \times 10^{-3} = 1261$ kN

The maximum shear force is at point 6 for load case 3.

$F_v \quad = F_{v\,@\,6} \quad = 593$ kN

593 kN $<$ 1261 kN

Therefore, the shear capacity is adequate.

8.2.5 Check the moment capacity

Basic requirement $M_x \leq M_{cx}$ 4.2.5.2

Check whether the shear is "high" (i.e. $F_v > 0.6\ P_v$) or "low" at the point of maximum moment. The maximum moment occurs at point 3 in load case 4, where:

$F_v \quad = 591$ kN.

$0.6\ P_v \quad = 0.6 \times 1261 \quad = 757$ kN

591 kN $<$ 757 kN

Therefore, **the shear is low**.

For low shear, the moment capacity for a class 1 section is given by: 4.2.5.2

$M_{cx} \quad = p_y\ S_x$

$M_{cx} \quad = 265 \times 3990 \times 10^{-3} \; = \; 1057$ kNm

Check limit to avoid irreversible deformation under serviceability loads. 4.2.5.1

For a continuous beam $M_{cx} \leq 1.5\ p_y Z_x$

$1.5 p_y Z_x \quad = 1.5 \times 265 \times 3480 \times 10^{-3} \; = \; 1383$ kNm

Therefore $\quad M_{cx} \quad = 1057$ kNm

Maximum moment is at point 3 for load case 4.

$M_x \quad = M_{x\,@\,3} \; = 1029$ kNm

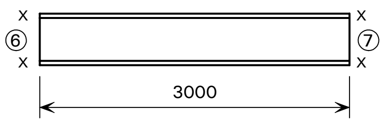
1029 kNm $<$ 1057 kNm

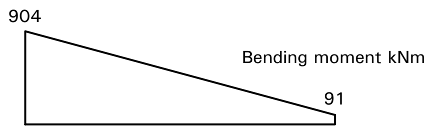
Therefore, **the moment capacity is adequate.**

8.2.6 Lateral-torsional buckling

As the top flange is fully restrained, lateral-torsional buckling will only be critical when the beam is subject to a hogging moment with the bottom flange in compression. By inspection of Figures 8.4 (a) to (d), it can be seen that the most critical part of the beam will be either the section between points 6 and 7 for load case 1, or the section between points 2 and 3 for load case 4.

Check between points 6 and 7 in load case 1





Basic requirements: $\quad \boldsymbol{M_x \leq M_b / m_{LT}}$ 4.3.6.2

and $\quad \boldsymbol{M_x \leq M_{cx}}$

(already checked in 8.2.5)

The buckling resistance moment M_b for a class 1 plastic section is given by:

$M_b \quad = p_b\ S_x$ 4.3.6.4

where p_b is the bending strength and is dependent on the design strength p_y and the equivalent slenderness λ_{LT}.

Example 8 Continuous beam designed elastically	Sheet 7 of 9	Rev

$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$ 4.3.6.7

$$\lambda = \frac{L_E}{r_y} = \frac{3000}{52.4} = 57$$

$$\lambda/x = \frac{57}{43.8} = 1.3$$

For a section with equal flanges and $\lambda/x = 1.3$, 4.3.6.7
$v = 0.98$ Table 19

For a class 1 plastic section, $\beta_W = 1.0$ 4.3.6.9

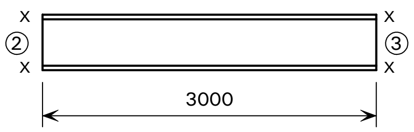
Therefore, $\lambda_{LT} = 0.863 \times 0.98 \times 57 \times 1.0 = 48.2$ 4.3.6.5
For $p_y = 265$ N/mm^2 and $\lambda_{LT} = 48.2$, Table 16 gives $p_b = 235$ N/mm^2 Table 16
$M_b = 235 \times 3990 \times 10^{-3} = 938$ kNm

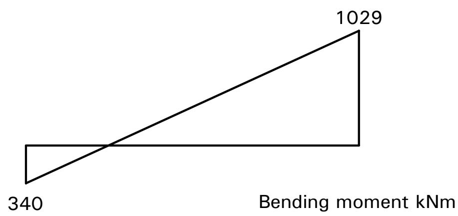
Since there is no loading between the points of intermediate restraint, the value of m_{LT} is obtained from the top part of Table 18 (i.e. segments with end moments only). 4.3.6.6 Table 18

$$\beta = \frac{91}{904} = 0.1$$

From Table 18, $m_{LT} = 0.64$
$M_b/m_{LT} = 938 / 0.64 = 1466$ kNm
$M_x = 1029$ kNm
1029 kNm < 1466 kNm
Therefore, **the buckling resistance moment between 6 and 7 in load case 1 is adequate.**

Check between points 2 and 3 in load case 4





Basic requirements: $\boldsymbol{M_x \le M_b/m_{LT}}$ 4.3.6.2
$\boldsymbol{M_x \le M_{cx}}$
(already checked in 8.2.5)

The buckling resistance moment M_b is already known from the calculation between points 6 and 7, so all that is required is a new value for m_{LT}.

Example 8 Continuous beam designed elastically	Sheet 8 of 9	Rev

$\beta = \dfrac{-340}{1029} = -0.33$ — 4.3.6.6 Table 18

From Table 18, $m_{LT} = 0.47$

$M_b / m_{LT} = 938 / 0.47 = 1996$ kNm

$M_x = 1029$ kNm

1029 kNm < 1996 kNm

Therefore, **the buckling resistance moment between 2 and 3 in load case 4 is adequate.**

8.2.7 Web bearing and buckling

The web should be checked for bearing and buckling as in Example 2.

8.2.8 SLS deflection check

The deflections under unfactored imposed loads should be checked as in Example 2

Adopt 686 x 254 x 125 UB in grade S275

8.3 Blue Book approach

The member capacities and resistances calculated in Section 8.2 could have been obtained directly from Volume 1[2].

Try 686 x 254 x 125 UB in grade S275

8.3.1 Shear capacity

Vol 1
Page C-78

P_v = 1260 kN
F_v = $F_{v\,@\,6}$ = 593 kN
593 kN < 1260 kN
Therefore, **the shear capacity is adequate.**

8.3.2 Moment capacity

Vol 1
Page C-59

M_{cx} = 1060 kNm
M_x = $M_{x\,@\,3}$ = 1029 kNm
1029 kNm < 1060 kNm
Therefore, **the moment capacity is adequate.**

8.3.3 Buckling resistance moment

Vol 1
Page C-59

For L_E = 3.0 m, M_b = 938 kNm

Note

m_{LT} values are not included in Volume 1 and must be obtained from Table 18 of BS 5950-1:2000.

Check between points 6 and 7 in load case 1

From Table 18, m_{LT} = 0.64
M_b / m_{LT} = 938 / 0.64 = 1466 kNm
M_x = 1029 kNm
1029 kNm < 1466 kNm
Therefore, **the buckling resistance moment between 6 and 7 in load case 1 is adequate.**

Check between points 2 and 3 in load case 4

From Table 18, m_{LT} = 0.47
M_b / m_{LT} = 938 / 0.47 = 1996 kNm
M_x = 1029 kNm
1029 kNm < 1996 kNm
Therefore, **the buckling resistance moment between 2 and 3 in load case 4 is adequate.**

Adopt 686 x 254 x 125 UB in grade S275

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CALCULATION SHEET

Job No.	CDS 153	Sheet 1 of 8	Rev
Job Title	Example no. 9		
Subject	Continuous beam designed plastically		
Client	SCI	Made by MDH	Date Jun 2003
		Checked by ASM	Date Oct 2003

9 Continuous beam designed plastically

9.1 Introduction

The continuous non-composite beam shown below has the same geometry, loading and restraint conditions as the beam designed elastically in Example 8. Its top flange is fully restrained laterally by a composite slab supported on secondary beams, while the bottom flange is restrained at the supports and at the points of load application by the secondary beams. Design the beam plastically in S275 steel using a uniform section throughout.

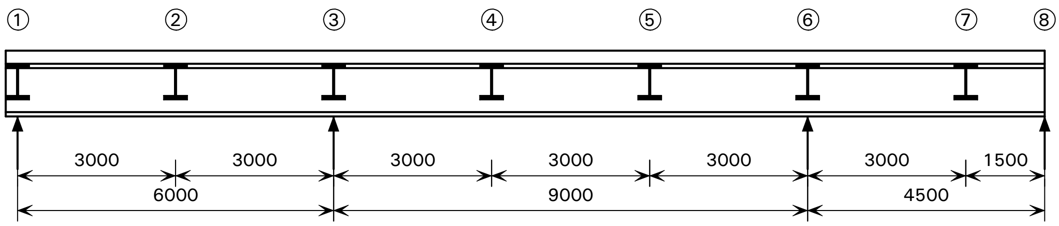

Figure 9.1

9.1.1 Loading

Taking the self weight of the beam as concentrated at the secondary beams, the bending moments and shear forces in the beam are related to the point loads at points 2, 4, 5 and 7 as shown in Figure 9.2.

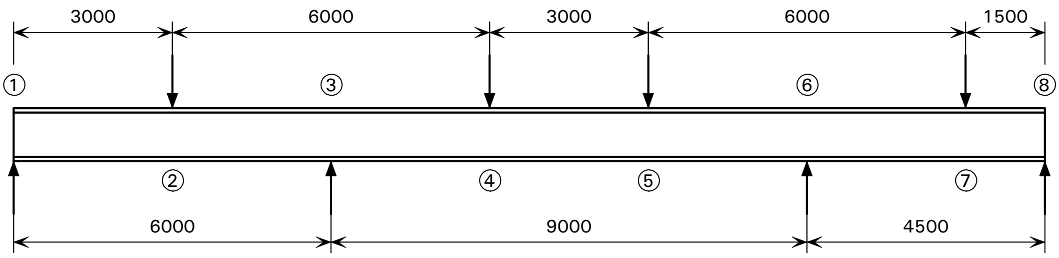

Figure 9.2

	Unfactored loads (kN)		Factored loads (kN)	
Location	Dead load	Imposed load	Dead load	Imposed load
2	150	225	210	360
4	150	225	210	360
5	150	225	210	360
7	150	300	210	480

Example 9 Continuous beam designed plastically	Sheet 2 of 8	Rev

Consider load case 1, i.e. (1.4 × dead load) + (1.6 × imposed load) on span 3-6, with (1.4 × dead load) elsewhere (see Figure 8.3 (a) and Figure 8.4 (a) in Example 8). This is the worst case for moment capacity in span 3-6 and for buckling resistance in spans 1-3 and 6-8.

9.1.2 Plastic hinges

Assume plastic hinges at points 3,4,5 and 6. The bending moments and shear forces are then as shown in Figure 9.3.

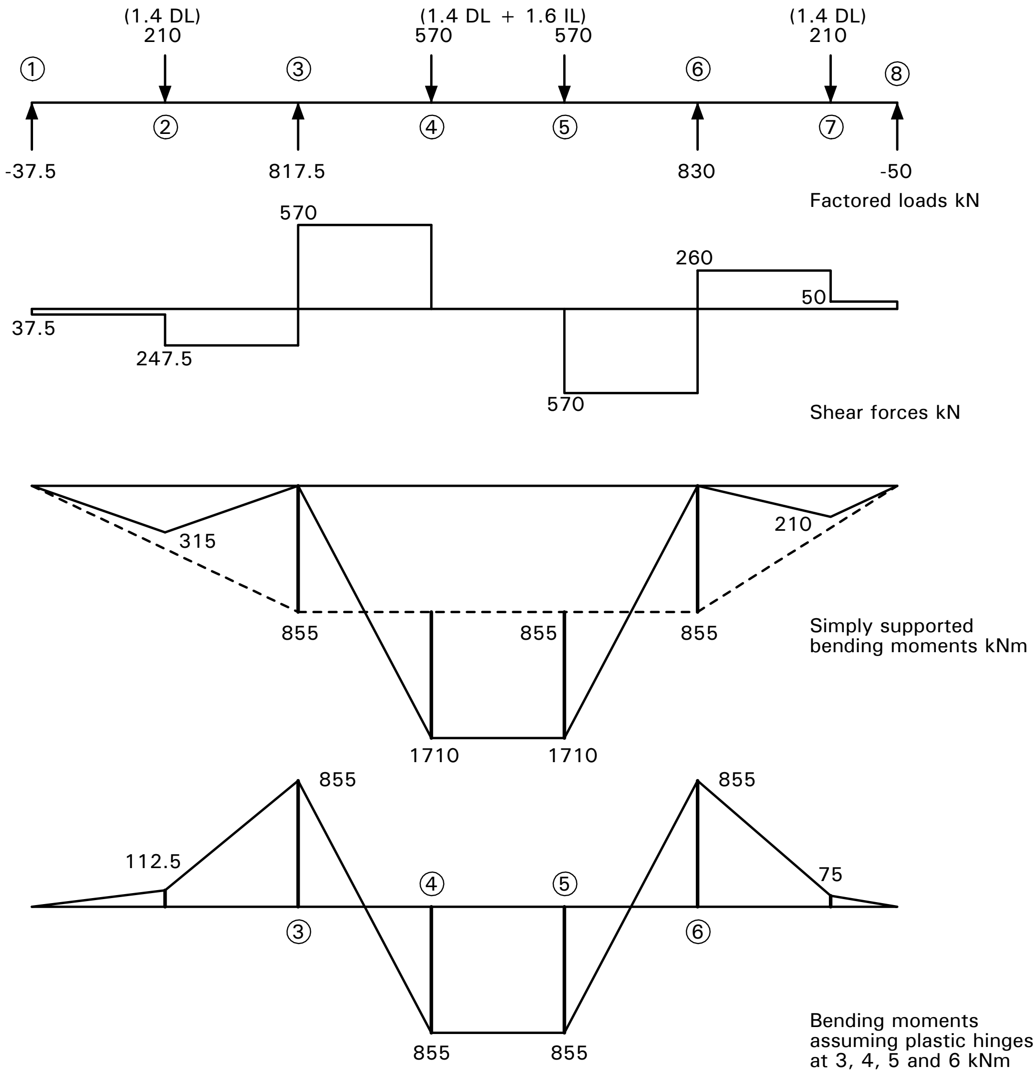

Figure 9.3

By inspection, span 3-6 is the critical one.

Total free moment $M_f = 570 \times 3 = 1710$ kNm.

$$M_p = \frac{M_f}{2} = \frac{1710}{2} = 855 \text{ kNm}$$

Assuming a design strength p_y = 265 N/mm^2, the required plastic modulus S_{req} is given by:

$$S_{req} = \frac{M_p}{p_y} = \frac{855}{265} \times 10^3 = 3226 \text{ cm}^3$$

9.2 Member design

9.2.1 Trial section

Try 610 x 229 x 113 UB in grade S275

From section property tables: (Vol 1 Page B-2; Vol 1 Page B-3)

Depth	D	= 607.6 mm
Width	B	= 228.2 mm
Web thickness	t	= 11.1 mm
Flange thickness	T	= 17.3 mm
Depth between fillets	d	= 547.6 mm
Radius of gyration	r_y	= 4.88 cm
Plastic modulus	S_x	= 3280 cm^3
Elastic modulus	Z_x	= 2870 cm^3
Torsional index	x	= 38.1

Local buckling ratios: (Vol 1 Page B-2)

Flange	b/T	= 6.60
Web	d/t	= 49.3

9.2.2 Classify the cross section

Grade of steel = S275 (3.1.1 Table 9)

T > 16.0 mm

Therefore p_y = 265 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{265}} = 1.02$$

For the outstand element of a compression flange, the limiting b/T for a class 1 plastic flange is 9ε. (3.5.2 Table 11)

Limiting b/T = 9ε = 9×1.02 = 9.18

The actual b/T = 6.60 < 9.18

Therefore, **the flange is class 1**.

Since the section is symmetrical and subject to pure bending, the neutral axis is at mid depth. For this case, the limiting d/t for a class 1 plastic web is 80ε. (3.5.2 Table 11)

Limiting d/t = 80ε = 80×1.02 = 81.6

The actual d/t = 49.3 < 81.6

Therefore, the **web is class 1**.

The cross section is class 1 and is, therefore, **suitable for plastic design**.

9.2.3 Check for shear buckling

If the d/t ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000[1]. (4.2.3)

70ε = 70×1.02 = 71.4

In this case, d/t = 49.3 < 70ε, so **there is no need to check for shear buckling**.

Example 9 Continuous beam designed plastically	Sheet 4 of 8	Rev

9.2.4 Check the shear capacity

Basic requirement $F_v \leq P_v$ (4.2.3)

$P_v = 0.6\, p_y\, A_v$

$p_y = 265$ N/mm^2

$A_v = tD = 11.1 \times 607.6 = 6744$ mm^2

$P_v = 0.6 \times 265 \times 6744 \times 10^{-3} = 1072$ kN

$F_v = 570$ kN

570 kN < 1072 kN

Therefore, **the shear capacity is adequate.**

9.2.5 Check the moment capacity

Basic requirement $M_x \leq M_{cx}$

Check whether the shear is "high" (i.e. $F_v > 0.6\, P_v$) or "low" at the point of maximum moment. The maximum moment and the maximum shear occur at the same location, i.e. $F_v = 570$ kN. (4.2.5.2)

$0.6\, P_v = 0.6 \times 1072 = 643$ kN

570 kN < 643 kN

Therefore, **the shear is low**.

For low shear, the moment capacity for a class 1 section is given by: (4.2.5.2)

$M_{cx} = p_y\, S_x$

$M_{cx} = 265 \times 3280 \times 10^{-3} = 869$ kNm

Check limit to avoid irreversible deformation under serviceability loads. (4.2.5.1)

For a continuous beam $M_{cx} \leq 1.5\, p_y Z_x$

$1.5 p_y Z_x = 1.5 \times 265 \times 2870 \times 10^{-3} = 1141$ kNm

Therefore $M_{cx} = 869$ kNm

Maximum moment $M_x = M_p = 855$ kNm.

855 kNm < 869 kNm

Therefore, **the moment capacity is adequate.**

9.3 Restraints

9.3.1 Restraints at plastic hinges

Both flanges should be laterally restrained at each plastic hinge location or within a distance of $D/2$ along the length of the member from the plastic hinge location. (5.3.2)

9.3.2 Restraints adjacent to a plastic hinge

Lateral restraint to the compression flange should be provided in accordance with one of the four methods given below. Method 1 is the most conservative and method 4 is the most rigorous. (5.3.3)

Method 1 – Ignoring restraint to the top flange and assuming a uniform moment (5.3.3 (a))

The maximum distance from the restraint at a hinge to an adjacent restraint, L_m, may be taken as equal to L_u given by:

$$L_u = \frac{38 r_y}{\left[\frac{f_c}{130} + \left(\frac{x}{36}\right)^2 \left(\frac{p_y}{275}\right)^2\right]^{0.5}}$$

Example 9 Continuous beam designed plastically	Sheet 5 of 8	Rev

f_c, is the average compressive stress in N/mm² due to the applied axial load. In this example, the beam is subjected to bending only, so $f_c = 0$.

$$L_m = L_u = \frac{38 \times 48.8}{\left[\left(\frac{38.1}{36}\right)^2 \left(\frac{265}{275}\right)^2\right]^{0.5}} = 1818 \text{ mm}$$

In this case, an additional restraint must be provided within 1818 mm of each plastic hinge position. This can be achieved by providing bottom flange restraints at the mid-points of parts 2-3, 3-4, 5-6 and 6-7.

Method 2 – Ignoring restraint to the top flange and allowing for moment gradient 5.3.3(b)

Note: This method should only be used for uniform I-section members with equal flanges and $D/B \geq 1.2$, in S275 or S355 steel where f_c does not exceed 80 N/mm².

Method 1 assumes a uniform bending moment in the beam between the plastic hinge position and the location of the next restraint. Method 2 uses an improved value of L_m by making an approximate allowance for the moment gradient. In this case, L_m is given by:

$$L_m = \phi L_u$$

where ϕ is dependent on the end moment ratio β and the limiting ratio β_u.

The critical segment length is that between points 2 and 3, where the bending moments are as shown below.

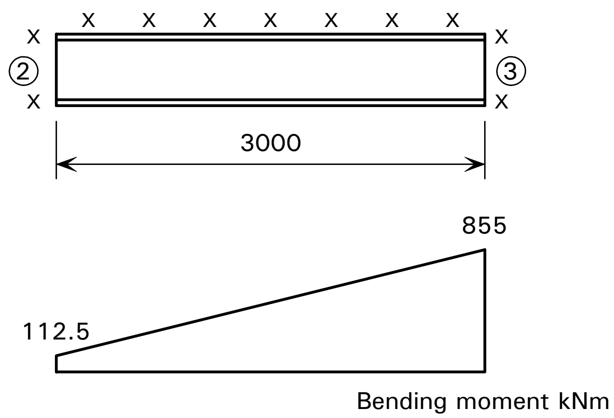

Figure 9.4 *Segment 2-3*

$$\beta = \frac{112.5}{855} = 0.132$$

For S275 steel, β_u is given by:

$$\beta_u = 0.44 + \frac{x}{270} - \frac{f_c}{200} = 0.44 + \frac{38.1}{270} - 0 = 0.581$$

Example 9 Continuous beam designed plastically	Sheet 6 of 8	Rev

For $\beta_u > \beta > 0$, ϕ is given by:

$\phi = 1 - (1 - KK_0)(\beta_u - \beta)/\beta_u$

$K_0 = (180 + x)/300 = (180 + 38.1)/300 = 0.727$

For $30 \le x \le 50$

$K = 0.8 + 0.08x - (x-10)f_c/2000 = 0.8 + (0.08 \times 38.1) = 3.848$

Therefore, $\phi = 1 - (1 - (3.848 \times 0.727))(0.581 - 0.132)/0.581 = 2.389$

$L_m = \phi L_u = 2.389 \times 1818 = 4343$ mm.

Using this method, no intermediate restraints to the compression flange are needed.

Method 3 – allowing for restraint to the top flange and ignoring moment gradient 5.3.4

Advantage may be taken of the restraint to the tension flange near the supports, by using the method given in Annex G (see method 4) or the simplified approach in clause 5.3.4. The simplified approach should only be used for I-section members with $D/B \ge 1.2$ in S275 or S355 steel.

In the simple method, the limiting spacing L_s for S275 steel is given conservatively by:

$$L_s = \frac{620 r_y}{K_1 \left[72 - \left(\frac{100}{x}\right)^2\right]^{0.5}}$$

For an un-haunched segment, $K_1 = 1.0$

$$L_s = \frac{620 \times 48.8}{\left[72 - \left(\frac{100}{38.1}\right)^2\right]^{0.5}} = 3750 \text{ mm}$$

Using this method, no intermediate restraints to the compression flange are needed.

Method 4 – allowing for restraint to the top flange and allowing for moment gradient G.3.3.

For uniform members or segments with a linear moment gradient (as in this case) the limiting spacing L_s between points of restraint to the compression flange is given by:

$$L_s = \frac{L_k}{\sqrt{m_t}}\left(\frac{M_{px}}{M_{rx} + aF_c}\right)^{0.5}$$ G.3.3.1

Since the axial load $F_c = 0$, $M_{rx} = M_{px}$ and the criterion becomes:

$$L_s = \frac{L_k}{\sqrt{m_t}}$$

Example 9 Continuous beam designed plastically	Sheet 7 of 8	Rev

The limiting length L_k for a uniform member subject to uniform moment, for an I-section with $x \geq 20$ and $D \geq 1.2B$ is obtained from:

$$L_k = \frac{\left(5.4 + 600\dfrac{p_y}{E}\right) r_y x}{\left(5.4x^2\dfrac{p_y}{E} - 1\right)^{0.5}} = \frac{\left(5.4 + 600 \times \dfrac{265}{205000}\right) 48.8 \times 38.1}{\left(5.4(38.1)^2 \times \dfrac{265}{205000} - 1\right)^{0.5}} = 3799 \text{ mm}$$

G.3.3.3

The equivalent uniform moment factor m_t is obtained from Table G.1 for given values of y and β_t.

$$\beta_t = \frac{112.5}{855} = 0.132$$

G.4.2

$$y = \left[\frac{1 + (2a/h_s)^2}{1 + (2a/h_s)^2 + 0.05(\lambda/x)^2}\right]^{0.5}$$

G.2.3

$$a = \frac{D}{2} = \frac{607.6}{2} = 303.8 \text{ mm}$$

Figure G.1

$$h_s = D - T = 607.6 - 17.3 = 590.3 \text{ mm}$$

$$\frac{2a}{h_s} = \frac{2 \times 303.8}{590.3} = 1.03$$

$$\lambda = \frac{L_y}{r_y} = \frac{3000}{48.8} = 61.5$$

$$\frac{\lambda}{x} = \frac{61.5}{38.1} = 1.61$$

$$y = \left[\frac{1 + (1.03)^2}{1 + (1.03)^2 + 0.05(1.61)^2}\right]^{0.5} = 0.97$$

For $\beta_t = 0.132$ and $y = 0.97$, $m_t = 0.58$

Table G.1

$$L_s = \frac{L_k}{\sqrt{m_t}} = \frac{3799}{\sqrt{0.58}} = 4988 \text{ mm}$$

G.3.3.1

Using this method, no intermediate restraints to the compression flange are needed.

9.4 Stiffeners

9.4.1 At hinge locations

5.2.3.7

Web stiffeners should be provided where a force that exceeds 10% of the shear capacity of the cross-section is applied to the web within a distance $D/2$ of a plastic hinge.

Consider hinge position 4:

Shear capacity $P_v = 1072$ kN

Applied load $F = 570$ kN $= 53\%$ of the shear capacity.

Therefore, **a stiffener would be required in this case.**

9.4.2 At supports and points of load application (not hinges)

The need for stiffeners at points 1, 2, 7 and 8 should be assessed using the same procedure as in Example 2. At each location, the load case giving the maximum reaction should be considered.

Adopt 610 x 229 x 113 UB in S275 steel

Comparing Examples 7 and 8, it is apparent that plastic design produces greater economy in terms of the weight of steel required and also reduces the depth of construction.

Note: The deflections under unfactored imposed loads should be checked as in Example 2.

9.5 Blue Book approach

The member capacities calculated in section 9.2 could have been obtained directly from Volume 1[2].

Try 610 x 229 x 113 UB in grade S275

9.5.1 Shear capacity

Vol 1 Page C-78

$P_v = 1070$ kN

$F_v = 570$ kN

570 kN < 1070 kN

Therefore, **the shear capacity is adequate.**

9.5.2 Moment capacity

Vol 1 Page C-59

$M_{cx} = 869$ kNm

Maximum moment $M_x = M_p = 855$ kNm.

855 kNm < 869 kNm

Therefore, **the moment capacity is adequate.**

Note: Volume 1 does not consider the issue of restraint adjacent to plastic hinges.

Adopt 610 x 229 x 113 UB in S275 steel

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CALCULATION SHEET

Job No. CDS 153		Sheet 1 of 3	Rev
Job Title Example no. 10			
Subject Pinned column using a non-slender UC			
Client SCI	Made by MDH	Date Jun 2003	
	Checked by ASM	Date Oct 2003	

10 Pinned column using a non-slender UC

10.1 Introduction

The column shown in Figure 10.1 is pin-ended about both axes and has no intermediate restraint. Design the column in S275 steel for the factored force shown below.

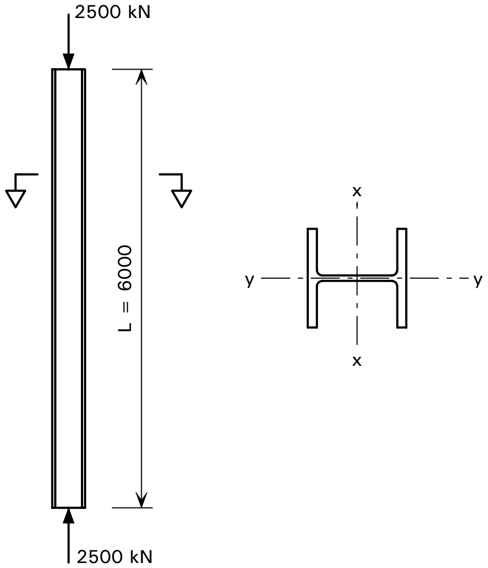

Figure 10.1

10.2 Member checks

10.2.1 Determine the effective length

The member is effectively held in position at both ends, but not restrained in direction at either end. Therefore, the effective length $L_E = 1.0L = 6000$ mm. — 4.7.3, Table 22

10.2.2 Trial section

Try 356 x 368 x 129 UC in grade S275

From section property tables: — Vol 1, Page B-8

Depth	D	= 355.6 mm
Width	B	= 368.6 mm
Web thickness	t	= 10.4 mm
Flange thickness	T	= 17.5 mm
Depth between fillets	d	= 290.2 mm
Radius of gyration about x-x axis	r_x	= 15.6 cm
Radius of gyration about y-y axis	r_y	= 9.43 cm
Gross area	A_g	= 164 cm^2

Vol 1, Page B-9

Example 10 Pinned column using a non-slender UC	Sheet 2 of 3	Rev

Local buckling ratios:
Flange b/T = 10.5
Web d/t = 27.9

Vol 1
Page B-8

10.2.3 Section classification

Grade of steel = S275
T > 16 mm and < 40 mm
Therefore, design strength p_y = 265 N/mm^2

3.1.1
Table 9

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{265}} = 1.02$$

In the design of columns under axial compression only, it is sufficient to determine whether the section is "class 4 slender" or "not class 4 slender".

4.7.4

For the outstand element of a compression flange, the limiting b/T for a class 3 semi-compact flange is 15ε.
Limiting b/T = 15ε = 15 × 1.02 = 15.3
The actual b/T = 10.5 < 15.3
Therefore, **the flange is "not class 4 slender".**

3.5.2
Table 11

For the web of an I-or H-section under axial compression, the limiting d/t for a class 3 semi-compact web is $\dfrac{120\varepsilon}{1+2r_2}$ but ≥ 40ε

3.5.2
Table 11

$$r_2 = \frac{F_c}{A_g p_y} = \frac{2500000}{16400 \times 265} = 0.58$$

$$\text{Limiting } d/t = \frac{120 \times 1.02}{1+(2 \times 0.58)} = 56.7$$

The actual d/t = 27.9 < 56.7
Therefore, **the web is "not class 4 slender".**

The cross section is "not class 4 slender".

10.2.4 Slenderness

4.7.2

$$\lambda_x = \frac{L_{Ex}}{r_x} = \frac{6000}{156} = 38.5$$

$$\lambda_y = \frac{L_{Ey}}{r_y} = \frac{6000}{94.3} = 63.6$$

10.2.5 Check the compression resistance

Basic requirement $F_c \le P_c$
P_c = $A_g p_c$ (for a class 4 non-slender cross section)
A_g = 164 cm^2

4.7.4

Example 10 Pinned column using a non-slender UC	Sheet 3 of 3	Rev

The compressive strength p_c is obtained from the relevant strut curve for buckling about the x-x and y-y axes. For a rolled H-section with $T < 40$ mm:

Buckling about the x-x axis

Use strut curve *b* — Table 23

For $\lambda_x = 38.5$ and $p_y = 265$ N/mm^2, $p_{cx} = 243$ N/mm^2 — Table 24b

Buckling about the y-y axis

Use strut curve *c* — Table 23

For $\lambda_y = 63.6$ and $p_y = 265$ N/mm^2, $p_{cy} = 189$ N/mm^2 — Table 24c

Therefore, $p_c = 189$ N/mm^2

$P_c = 16400 \times 189 \times 10^{-3} = 3100$ kN

$F_c = 2500$ kN

2500 kN < 3100 kN

Therefore, **the compression resistance is adequate.**

Adopt 356 x 368 x 129 UC in S275

10.3 Blue Book approach

The compression resistance calculated above could have been obtained directly from Volume 1[2]. — Vol 1 Page C-6

Try 356 x 368 x 129 UC in grade S275

For $L_E = 6.0$ m, $P_{cx} = 3990$ kN and $P_{cy} = 3090$ kN

Therefore, $P_c = 3090$ kN

$F_c = 2500$ kN

2500 kN < 3090 kN

Therefore, **the compression resistance is adequate.**

Adopt 356 x 368 x 129 UC in S275

The Steel Construction Institute Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 623345 Fax: (01344) 622944 **CALCULATION SHEET**	Job No. CDS 153		Sheet 1 of 2		Rev
	Job Title Example no. 11				
	Subject Pinned column using a non-slender RHS				
	Client SCI	Made by MDH		Date Jun 2003	
		Checked by ASM		Date Oct 2003	

11 Pinned column using a non-slender RHS

11.1 Introduction

Redesign the column shown in Example 10 using a grade 355 hot finished square hollow section.

Design axial compressive force F_c = 2500 kN.

11.2 Member checks

11.2.1 Determine the effective length

4.7.3
Table 22

The member is effectively held in position at both ends, but not restrained in direction at either end. Therefore, the effective length $L_E = 1.0L = 6000$ mm.

11.2.2 Trial section

Try 250 x 250 x 10 HF RHS (74.5 kg/m) in S355

From section property tables:

Vol 1
Page B-22

Depth	D	= 250.0 mm
Thickness	t	= 10.0 mm
Radius of gyration	r	= 9.77 cm
Gross area	A_g	= 94.9 cm^2
Local buckling ratio	d/t	= 22.0

11.2.3 Section classification

3.1.1
Table 9

Grade of steel = S355

$t < 16$ mm

Therefore, design strength p_y = 355 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{355}} = 0.88$$

4.7.4

In the design of columns under axial compression only, it is sufficient to determine whether the section is “class 4 slender” or “not class 4 slender”.

3.5.2
Table 12

The limiting d/t for a class 3 semi-compact hot-finished RHS under axial compression is 40ε.

Limiting d/t = 40ε = 40×0.88 = 35.2

The actual d/t = $22.0 < 35.2$

Therefore, **the cross section is “not class 4 slender”.**

Example 11 Pinned column using a non-slender RHS	Sheet 2 of 2	Rev

11.2.4 Slenderness

$$\lambda = \frac{L_E}{r} = \frac{6000}{97.7} = 61.4$$ 4.7.2

11.2.5 Check the compression resistance

Basic requirement $F_c \leq P_c$ 4.7.4

$P_c = A_g p_c$ (for a non-slender cross section)

$A_g = 94.9\ \text{cm}^2$

The compressive strength p_c is obtained from the relevant strut curve. Table 23
For hot-finished hollow sections, use strut curve a.
(Note that for cold-formed hollow sections, strut curve c should be used).

For $\lambda = 61.4$ and $p_y = 355\ \text{N/mm}^2$, $p_c = 294\ \text{N/mm}^2$ Table 24a

$P_c = 9490 \times 294 \times 10^{-3} = 2790\ \text{kN}$
$F_c = 2500\ \text{kN}$
2500 kN < 2790 kN
Therefore, **the compression resistance is adequate.**

Adopt 250 x 250 x 10 HF RHS (74.5 kg/m) in grade S355

11.3 Blue Book approach

The compression resistance calculated above could have been obtained directly from Volume 1[2].

Try 250 x 250 x 10 HF RHS (74.5 kg/m) in S355

For $L_E = 6.0$ m, $P_c = 2790\ \text{kN}$ Vol 1
$F_c = 2500\ \text{kN}$ Page D-12
2500 kN < 2790 kN
Therefore, **the compression resistance is adequate.**

Adopt 250 x 250 x 10 HF RHS (74.5 kg/m) in grade S355

The Steel Construction Institute Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 623345 Fax: (01344) 622944 **CALCULATION SHEET**	Job No. CDS 153	Sheet 1 of 2	Rev
	Job Title Example no. 12		
	Subject Pinned columnn using a slender CHS		
	Client SCI	Made by MDH	Date Jun 2003
		Checked by ASM	Date Oct 2003

12 Pinned column using a slender CHS

12.1 Introduction

Redesign the column shown in Example 10 using a grade S355 hot finished circular hollow section.

Design axial compressive force F_c = 2500 kN.

12.2 Member checks

12.2.1 Determine the effective length

The member is effectively held in position at both ends, but not restrained in direction at either end. Therefore, the effective length $L_E = 1.0L = 6000$ mm. 4.7.3 Table 22

12.2.2 Trial section

Try 406.4 x 6.3 HF CHS (62.2 kg/m) in S355 Vol 1 Page B-20

From section property tables:

Outside diameter	D	= 406.4 mm
Thickness	t	= 6.3 mm
Radius of gyration	r	= 14.1 cm
Gross area	A_g	= 79.2 cm^2
Local buckling ratio	D/t	= 64.5

12.2.3 Section classification

Grade of steel = S355 3.1.1 Table 9

$t < 16$ mm

Therefore, design strength p_y = 355 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{355}} = 0.88$$

In the design of columns under axial compression only, it is sufficient to determine whether the section is "class 4 slender" or "not class 4 slender". 4.7.4

The limiting D/t for a class 3 semi-compact CHS under axial compression is $80\varepsilon^2$. 3.5.2 Table 12

Limiting $D/t = 80\varepsilon^2 = 80 \times 0.88^2 = 62.0$

The actual $d/t = 64.5 > 62.0$

Therefore, **the cross section is "class 4 slender".**

12.2.4 Slenderness

$$\lambda = \frac{L_E}{r} = \frac{6000}{141} = 42.6$$ 4.7.2

Example 12 Pinned column using a slender CHS	Sheet 2 of 2	Rev

12.2.5 Check the compression resistance

Basic requirement $F_c \leq P_c$ — 4.7.4

$P_c = A_{eff} p_{cs}$ (for a slender cross section)

$$A_{eff} = A\left[\left(\frac{80}{D/t}\right)\left(\frac{275}{p_y}\right)\right]^{0.5} = 79.2\left[\left(\frac{80}{64.5}\right)\left(\frac{275}{355}\right)\right]^{0.5} = 77.6\ \text{cm}^2$$ — 3.6.6

The compressive strength p_{cs} is obtained from the relevant strut curve using a reduced slenderness. — 4.7.4

Reduced slenderness $= 42.6 \times \left(\frac{77.6}{79.2}\right)^{0.5} = 42.2$

For hot-finished hollow sections, use strut curve a. — Table 23
(Note that for cold-formed hollow sections, strut curve c should be used).

For $\lambda = 42.2$ and $p_y = 355$ N/mm², $p_{cs} = 330$ N/mm² — Table 24a

$P_c = 7760 \times 330 \times 10^{-3} = 2561$ kN
$F_c = 2500$ kN
2500 kN < 2561 kN
Therefore, **the compression resistance is adequate.**

Adopt 406.4 x 6.3 HF CHS (62.2 kg/m) in S355

12.3 Blue Book approach

The buckling resistance calculated above could have been obtained directly from Volume 1[2].

Try 406.4 x 6.3 HF CHS (62.2 kg/m) in S355 — Vol 1 Page D-10

For $L_E = 6.0$ m, $P_c = 2560$ kN
$F_c = 2500$ kN
2500 kN < 2560 kN
Therefore, **the compression resistance is adequate.**

Adopt 406.4 x 6.3 HF CHS (62.2 kg/m) in S355

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	Job Title Example no. 13		
	Subject Pinned column with intermediate restraint		
	Client SCI	Made by MDH	Date Jun 2003
		Checked by ASM	Date Oct 2003

13 Pinned column with intermediate restraint

13.1 Introduction

The column shown in Figure 13.1 has the same length and axial force as in Example 10 with the addition of a tie at mid-height providing restraint about the y-y axis. Design the column in S275 steel.

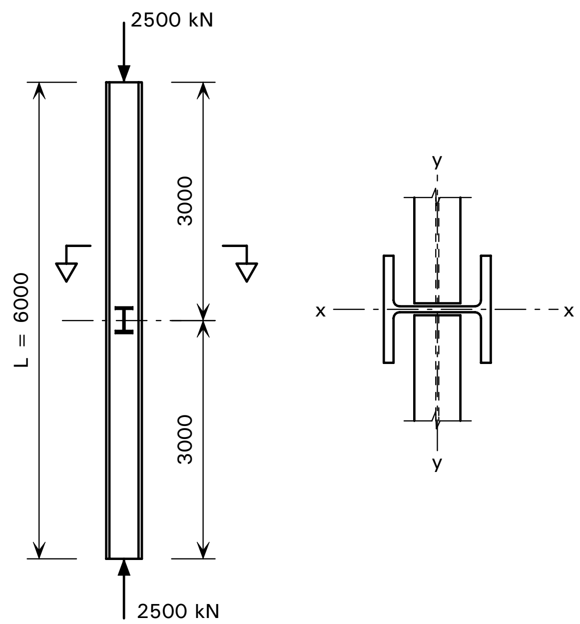

Figure 13.1

Design axial compressive force F_c = 2500 kN.

13.2 Member checks

13.2.1 Determine the effective lengths

4.7.3

The member is effectively held in position at both ends, but not restrained in direction at either end. The tie provides restraint in position only for buckling about the y-y axis (i.e. the member is not restrained in direction by the tie).

Table 22

About the x-x axis L_{Ex} = 1.0 × 6000 = 6000 mm.

About the y-y axis L_{Ey} = 1.0 × 3000 = 3000 mm.

13.2.2 Trial section

Try 305 x 305 x 97 UC in grade S275

From section property tables:

Depth	D	= 307.9 mm	Vol 1 Page B-8
Width	B	= 305.3 mm	
Web thickness	t	= 9.9 mm	
Flange thickness	T	= 15.4 mm	
Depth between fillets	d	= 246.7 mm	Vol 1 Page B-9
Radius of gyration about x-x axis	r_x	= 13.4 cm	
Radius of gyration about y-y axis	r_y	= 7.69 cm	
Gross area	A_g	= 123 cm^2	

Local buckling ratios:

Flange	b/T	= 9.91	Vol 1 Page B-8
Web	d/t	= 24.9	

13.2.3 Section classification

Grade of steel = S275 — 3.1.1 Table 9

$T < 16$ mm

Therefore, design strength $p_y = 275$ N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{275}} = 1.0$$

In the design of columns under axial compression only, it is sufficient to determine whether the section is "class 4 slender" or "not class 4 slender". — 4.7.4

For the outstand element of a compression flange, the limiting b/T for a class 3 semi-compact flange is 15ε. — 3.5.2 Table 11

Limiting $b/T = 15\varepsilon = 15 \times 1.0 = 15.0$

The actual $b/T = 9.91 < 15.0$

Therefore, **the flange is "not class 4 slender".**

For the web of an I-or H-section under axial compression, the limiting d/t for a class 3 semi-compact web is $\dfrac{120\varepsilon}{1+2r_2}$ but $\geq 40\varepsilon$ — 3.5.2 Table 11

$$r_2 = \frac{F_c}{A_g p_y} = \frac{2500000}{12300 \times 275} = 0.74$$

$$\text{Limiting } d/t = \frac{120 \times 1.0}{1 + (2 \times 0.74)} = 48.4$$

The actual $d/t = 24.9 < 48.4$

Therefore, **the web is "not class 4 slender".**

The cross section is "not class 4 slender".

13.2.4 Slenderness

4.7.2

$$\lambda_x = \frac{L_{Ex}}{r_x} = \frac{6000}{134} = 44.8$$

$$\lambda_y = \frac{L_{Ey}}{r_y} = \frac{3000}{76.9} = 39.0$$

13.2.5 Check the compression resistance

4.7.4

Basic requirement $F_c \leq P_c$

$P_c = A_g p_c$ (for a non-slender cross section)

$A_g = 123$ cm^2

The compressive strength p_c is obtained from the relevant strut curve for buckling about the x-x and y-y axes. For a rolled H-section with $T < 40$ mm:

Buckling about the x-x axis

Table 23
Table 24b

Use strut curve *b*

For $\lambda_x = 44.8$ and $p_y = 275$ N/mm^2, $p_{cx} = 244$ N/mm^2

Buckling about the y-y axis

Table 23
Table 24c

Use strut curve c

For $\lambda_y = 39.0$ and $p_y = 275$ N/mm^2, $p_{cy} = 240$ N/mm^2

Therefore, $p_c = 240$ N/mm^2

Note:

Although $\lambda_x > \lambda_y$, buckling about the y-y axis governs because the lower strut curve is used for buckling about the minor axis.

$P_c = 12300 \times 240 \times 10^{-3} = 2952$ kN

$F_c = 2500$ kN

2500 kN < 2952 kN

Therefore, **the compression resistance is adequate.**

Adopt 305 x 305 x 97 UC in S275

13.3 Blue Book approach

The buckling resistance calculated above could have been obtained directly from Volume 1[2].

Try 305 x 305 x 97 UC in grade S275

Vol 1
Page C-6

For $L_{Ex} = 6.0$ m, $P_{cx} = 3000$ kN

For $L_{Ey} = 3.0$ m, $P_{cy} = 2950$ kN

Therefore, $P_c = 2950$ kN

$F_c = 2500$ kN

2500 kN < 2950 kN

Therefore, **the compression resistance is adequate.**

Adopt 305 x 305 x 97 UC in S275

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CALCULATION SHEET

Job No. CDS 153		Sheet 1 of 5	Rev
Job Title Example no. 14			
Subject Continuous column in simple construction using a UC section			
Client SCI	Made by MDH	Date Jun 2003	
	Checked by ASM	Date Oct 2003	

14 Continuous column in simple construction using a UC section

14.1 Introduction

Design the column shown in Figure 14.1 in S275 steel between levels 1 and 2. Two solutions are presented; the full calculation and the Blue Book approach.

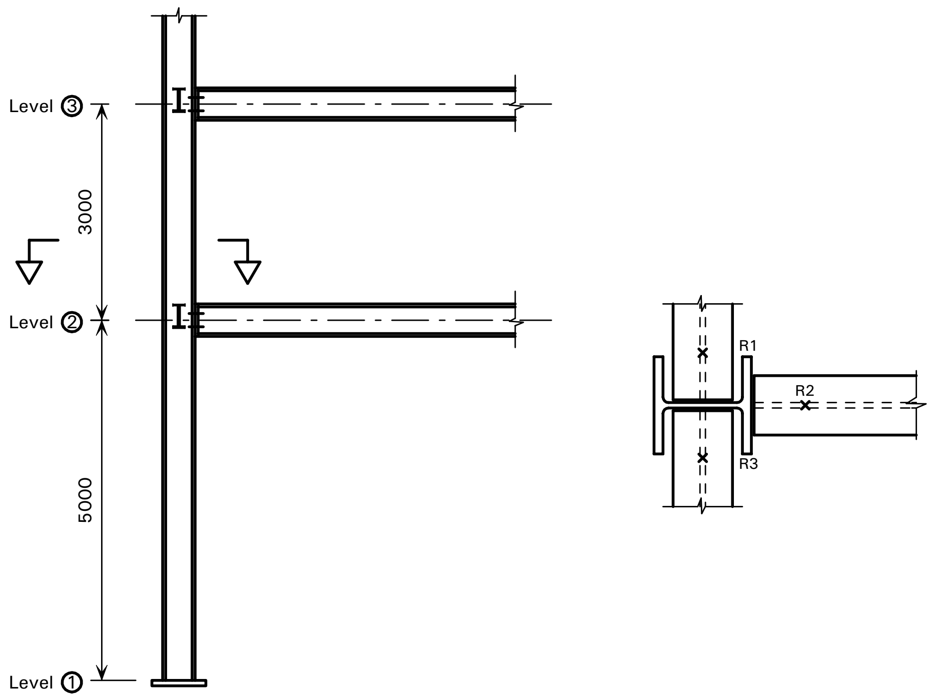

Figure 14.1

14.1.1 Assumptions

4.7.7

- The column is continuous and forms part of a structure of simple construction.
- The column is effectively pinned at the base.
- Beams are connected to the column flange by flexible end plates.

14.1.2 Loading (factored)

$F_{2\text{-}3}$ = 410 kN (axial force in column between levels 2 and 3)
R_1 = 40 kN (reaction from beam 1)
R_2 = 160 kN (reaction from beam 2)
R_3 = 30 kN (reaction from beam 3)

The total axial load acting on the column between levels 1 and 2 is given by:

$F_c = F_{2\text{-}3} + R_1 + R_2 + R_3$

$F_c = 410 + 40 + 160 + 30 = 640$ kN

14.2 Member checks

14.2.1 Determine the effective lengths

About the x-x axis, L_{Ex} = $0.85L$ = 0.85×5000 = 4250 mm — 4.7.3 Table 22

About the y-y axis, L_{Ey} = $1.0L$ = 1.0×5000 = 5000 mm

14.2.2 Trial section

The trial section may be obtained from the member capacity tables in Volume 1[2] by considering the axial force only (i.e. ignoring the moment due to eccentricity). This section should then be checked for the combined effects of axial force and nominal moment.

Try 203 x 203 x 46 UC in grade S275

From member capacity tables: — Vol 1 Page C-7

Compression resistance P_{cy} = 760 kN for L_E = 5.0 m

From section property tables:

Depth	D	= 203.2 mm	Vol 1 Page B-8
Width	B	= 203.6 mm	
Web thickness	t	= 7.2 mm	
Flange thickness	T	= 11.0 mm	
Depth between fillets	d	= 160.8 mm	Vol 1 Page B-9
Radius of gyration about x-x axis	r_x	= 8.82 cm	
Radius of gyration about y-y axis	r_y	= 5.13 cm	
Gross area	A_g	= 58.7 cm^2	
Plastic modulus about x-x axis	S_x	= 497 cm^3	
Elastic modulus about x-x axis	Z_x	= 450 cm^3	
Elastic modulus about y-y axis	Z_y	= 152 cm^3	

Local buckling ratios:

Flange	b/T	= 9.25	Vol 1 Page B-8
Web	d/t	= 22.3	

14.2.3 Section classification

Grade of steel = S275 — 3.1.1 Table 9

T < 16 mm

Therefore, design strength p_y = 275 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{275}} = 1.0$$

For the outstand element of a compression flange, the limiting b/T for a class 1 plastic flange is 9ε. Limiting b/T = 9ε = 9×1.0 = 9.0 — 3.5.2 Table 11

The actual b/T = 9.25 > 9.0

Therefore, the flange is not plastic and needs to be checked to see whether it is compact or not.

For the outstand element of a compression flange, the limiting b/T for a class 2 compact flange is 10ε. Limiting b/T = 10ε = 10×1.0 = 10.0 — 3.5.2 Table 11

The actual b/T = 9.25 < 10.0

Therefore, **the flange is class 2 compact.**

Example 14 Continuous column in simple construction using UC	Sheet 3 of 5	Rev

For the web of an I-or H-section under axial compression and bending (the "generally" case in Table 11), the limiting *d/t* for a class 1 plastic web is — 3.5.2 Table 11

$$\frac{80\varepsilon}{1+r_1} \text{ but} \geq 40\varepsilon$$

$$r_1 = \frac{F_c}{dtp_y} = \frac{640000}{160.8 \times 7.2 \times 275} = 2.91, \text{ but } -1 < r_1 \leq 1, \text{ therefore } r_1 = 1.$$ — 3.5.5

$$\text{Limiting } d/t = \frac{80 \times 1.0}{1+1} = 40$$

The actual *d/t* = 22.3 < 40
Therefore, the **web is class 1 plastic.**

Since the flange is class 2 compact and the web is class 1 plastic**, the cross section is class 2 compact**.

14.2.4 Slenderness

$$\lambda_x = \frac{L_{Ex}}{r_x} = \frac{4250}{88.2} = 48.2$$ — 4.7.2

$$\lambda_y = \frac{L_{Ey}}{r_y} = \frac{5000}{51.3} = 97.5$$

14.2.5 Check the compression resistance

Basic requirement $F_c \leq P_c$
$P_c = A_g p_c$ (for a class 2 compact cross section) — 4.7.4
$A_g = 58.7$ cm^2

The compressive strength p_c is obtained from the relevant strut curve for buckling about the x-x and y-y axes.

Buckling about the x-x axis

Use strut curve b — Table 23
For $\lambda_x = 48.2$ and $p_y = 275$ N/mm^2, $p_{cx} = 239$ N/mm^2 — Table 24b

Buckling about the y-y axis

Use strut curve c — Table 23
For $\lambda_y = 97.5$ and $p_y = 275$ N/mm^2, $p_{cy} = 130$ N/mm^2 — Table 24c

Therefore, $p_c = 130$ N/mm^2

$p_c = 5870 \times 130 \times 10^{-3}$
$F_c = 640$ kN

640 kN < 763 kN
Therefore, **the compression resistance is adequate.**

Example 14 Continuous column in simple construction using UC	Sheet 4 of 5	Rev

14.2.6 Nominal moments due to eccentricity

For columns in simple construction, the beam reactions are assumed to act 100 mm off the face of the columns. 4.7.7

Total moment at level 2 from beam connected to column flange:

$$M_{2x} = R_2\ (D/2\ +\ 100) = 160\left(\frac{203.2}{2}+100\right)\times 10^{-3} = 32.3\ \text{kNm}$$

Total moment at level 2 from beams connected to column web:

$$M_{2y} = R_1\left(\frac{t}{2}+100\right)-R_3\left(\frac{t}{2}+100\right) = \left(R_1 - R_3\right)\left(\frac{t}{2}+100\right)$$

$$M_{2y} = \left(40-30\right)\left(\frac{7.2}{2}+100\right)\times 10^{-3} = 1.04\ \text{kNm}$$

These moments are distributed between the column lengths above and below level 2, in proportion to the bending stiffnesses of each length. 4.7.7

Stiffness of column 2 to 1 = $EI/5$
Stiffness of column 2 to 3 = $EI/3$

Assuming the same section size is used between levels 1 and 3, the ratio of these stiffness values is 5/3. Therefore, 3/8 of M_{2x} and M_{2y} are transferred into column 2-1 and 5/8 of M_{2x} and M_{2y} are transferred into column 2-3.

$$M_{x(2-1)} = \frac{3}{8}M_{2x} = 0.375\times 32.3 = 12.1\ \text{kNm}$$

$$M_{y(2-1)} = \frac{3}{8}M_{2y} = 0.375\times 1.04 = 0.39\ \text{kNm}$$

14.2.7 Combined axial force and moment check

Since only nominal moments are applied, the column should satisfy the relationship

$$\frac{F_c}{P_c}+\frac{M_x}{M_{bs}}+\frac{M_y}{p_y Z_y}\leq 1$$ 4.7.7

M_x = $M_{x(2\text{-}1)}$ = 12.1 kNm
M_y = $M_{y(2\text{-}1)}$ = 0.39 kNm

M_{bs} is the buckling resistance moment for simple columns. For I- and H sections, it should be taken as the value of M_b determined using an equivalent slenderness λ_{LT} given by:

λ_{LT} = 0.5 L/r_y
λ_{LT} = 0.5 (5000 / 51.3) = 48.7

Note: This expression for λ_{LT} uses the actual length L and not the effective length L_E.

For p_y = 275 N/mm^2 and λ_{LT} = 48.7, p_b = 241 N/mm^2 Table 16

Example 14 Continuous column in simple construction using UC	Sheet 5 of 5	Rev

For a class 2 compact cross section, M_b is given by:

$M_b = p_b S_x = 241 \times 497 \times 10^{-3} = 120$ kNm 4.3.6.4

Therefore, $M_{bs} = 120$ kNm.

$$\frac{640}{763} + \frac{12.1}{120} + \frac{0.39 \times 10^6}{275 \times 152 \times 10^3}$$

Therefore, **the combined resistance against axial force and moment is adequate.**

Adopt 203 x 203 x 46 UC in S275

14.3 Blue Book approach

The values of P_c, M_{bs} and $p_y Z_y$ could have been obtained directly from Volume 1[2].

Try 203 x 203 x 46 UC in grade S275

14.3.1 Compression resistance

For $L_E = 5.0$ m, $P_{cy} = 760$ kN Vol 1 Page C-131

Volume 1 does not give P_c values for $L_{Ex} = 4.25$ m, but the following values are given for $L_{Ex} = 4.0$ m and $L_{Ex} = 5.0$ m:

For $L_{Ex} = 4.0$m, $P_{cx} = 1430$ kN
For $L_{Ex} = 5.0$m, $P_{cx} = 1330$ kN

Clearly P_{cy} is critical, therefore $P_c = 760$ kN
$F_c = 640$ kN
640 kN < 760 kN
Therefore, **the compression resistance is adequate.**

14.3.2 Combined axial force and moment check

For $L = 5.0$ m, $M_{bs} = 120$ kNm Vol 1 Page C-131
$p_y Z_y = 41.8$ kNm

$$\frac{640}{760} + \frac{12.1}{120} + \frac{0.39}{41.8}$$

$= 0.842 + 0.101 + 0.009 = 0.95 < 1$

Therefore, **the combined resistance against axial force and moment is adequate.**

Adopt 203 x 203 x 46 UC in S275

Note: Volume 1 only provides values for the terms in the interaction formula. The interaction check itself must still be carried out by hand.

The Steel Construction Institute	Job No.	CDS 153	Sheet 1 of 5		Rev
	Job Title	Example no. 15			
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 623345 Fax: (01344) 622944	Subject	Continuous column in simple construction using an RHS			
	Client	SCI	Made by	MDH	Date Jun 2003
CALCULATION SHEET			Checked by	ASM	Date Oct 2003

15 Continuous column in simple construction using an RHS

15.1 Introduction

Redesign the column shown in Example 14 between levels 1 and 2 using a hot finished square hollow section in S355 (see Figure 15.1). Two solutions are presented; the full calculation and the Blue Book approach.

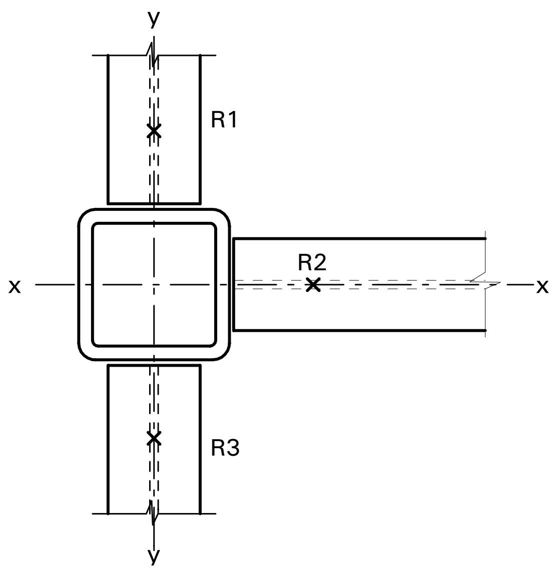

Figure 15.1

15.1.1 Assumptions

- The column is continuous and forms part of a structure of simple construction. 4.7.7
- The column is effectively pinned at the base.
- Beams are connected to the RHS column by flexible end plates.

15.1.2 Loading (factored)

$F_{2\text{-}3}$ = 410 kN (axial force in column between levels 2 and 3)
R_1 = 40 kN (reaction from beam 1)
R_2 = 160 kN (reaction from beam 2)
R_3 = 30 kN (reaction from beam 3)

The total axial load acting on the column between levels 1 and 2 is given by:

$F_c = F_{2\text{-}3} + R_1 + R_2 + R_3$

$F_c = 410 + 40 + 160 + 30 = 640$ kN

15.2 Member checks

15.2.1 Determine the effective lengths

About the x-x axis, $L_{Ex} = 0.85L = 0.85 \times 5000 = 4250$ mm
About the y-y axis, $L_{Ey} = 1.0L = 1.0 \times 5000 = 5000$ mm

4.7.3
Table 22

15.2.2 Trial section

The trial section may be obtained from the member capacity tables in Volume 1[2] by considering the axial force only (i.e. ignoring the moment due to eccentricity). This section should then be checked for the combined effects of axial force and moment.

Try 150 x 150 x 6.3 HF RHS (28.1 kg/m) in S355

From member capacity tables:
Compression resistance $P_c = 774$ kN for $L_E = 5.0$ m

Vol 1
Page D-12

From section property tables:

Depth $D = 150$ mm
Thickness $t = 6.3$ mm
Radius of gyration $r = 5.85$ cm
Gross area $A_g = 35.8$ cm^2
Plastic modulus $S = 192$ cm^3
Elastic modulus $Z = 163$ cm^3
Local buckling ratio $d/t = 20.8$
$d = D - 3t = 150 - (3 \times 6.3) = 131.1$ mm

Vol 1
Page B-22

15.2.3 Section classification

Grade of steel = S355
$t < 16$ mm
Therefore, design strength $p_y = 355$ N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{355}} = 0.88$$

3.1.1
Table 9

The limiting b/t for the flange of a class 1 plastic hot finished RHS is 28ε but $\leq 80\varepsilon - d/t$
Limiting $b/t = 28 \times 0.88 = 24.6$ which is less than $80 - 20.8 = 59.2$
The actual $b/t = d/t = 20.8 < 24.6$
Therefore, **the flange is class 1 plastic**.

3.5.2
Table 12

For the web of an RHS under axial compression and bending (the "generally" case in Table 12), the limiting d/t for a class 1 plastic web is

$$\frac{64\varepsilon}{1 + 0.6r_1} \text{ but } \geq 40\varepsilon$$

3.5.2
Table 12

$$r_1 = \frac{F_c}{2dtp_y} = \frac{640000}{2 \times 131.1 \times 6.3 \times 355} = 1.09 \text{, but } -1 < r_1 \leq 1 \text{, therefore } r_1 = 1.$$

3.5.5

$$\text{Limiting } d/t = \frac{64 \times 0.88}{1 + 0.6} = 35.2$$

The actual $d/t = 20.8 < 35.2$
Therefore, **the web is class 1 plastic.**

Therefore, **the cross section is class 1 plastic**.

Example 15 Continuous column in simple construction using an RHS	Sheet 3 of 5	Rev

Note: For an RHS, the limiting local buckling ratios for the flange are never greater than the equivalent limits for the web. Since, for the special case of a square hollow section, the ratios *b/t* and *d/t* are equal, it follows that the section classification will always be governed by the flange.

15.2.4 Slenderness

$$\lambda = \frac{L_E}{r} = \frac{5000}{58.5} = 85.5$$ 4.7.2

15.2.5 Check the compression resistance

Basic requirement $F_c \leq P_c$

$P_c = A_g p_c$ (for a class 1 plastic cross section) 4.7.4

$A_g = 35.8\ \text{cm}^2$

The compressive strength p_c is obtained from the relevant strut curve.

For hot-finished hollow sections, use strut curve a. Table 23

(Note that for cold-formed hollow sections, strut curve c should be used).

For $\lambda = 85.5$ and $p_y = 355\ \text{N/mm}^2$, $p_c = 216\ \text{N/mm}^2$ Table 24a

$P_c = 3580 \times 216 \times 10^{-3} = 773\ \text{kN}$

$F_c = 640\ \text{kN}$

640 kN < 773 kN

Therefore, **the compression resistance is adequate.**

15.2.6 Nominal moments due to eccentricity

For columns in simple construction, the beam reactions are assumed to act 100 mm off the face of the columns. 4.7.7

Total moment at level 2 from beam 2 connected to RHS column:

$$M_{2x} = R_2\,(D/2 + 100) = 160\left(\frac{150}{2} + 100\right) \times 10^{-3} = 28.0\ \text{kNm}$$

Total moment at level 2 from beams 1 and 3 connected to RHS column:

$$M_{2y} = R_1\left(\frac{D}{2} + 100\right) - R_3\left(\frac{D}{2} + 100\right) = (R_1 - R_3)\left(\frac{D}{2} + 100\right)$$

$$M_{2y} = (40 - 30)\left(\frac{150}{2} + 100\right) \times 10^{-3} = 1.75\ \text{kNm}$$

These moments are distributed between the column lengths above and below level 2, in proportion to the bending stiffnesses of each length. 4.7.7

Stiffness of column 2 to 1 = $EI/5$

Stiffness of column 2 to 3 = $EI/3$

Assuming the same section size is used between levels 1 and 3, the ratio of these stiffness values is 5/3. Therefore, 3/8 of M_{2x} and M_{2y} are transferred into column 2-1 and 5/8 of M_{2x} and M_{2y} are transferred into column 2-3.

Example 15 Continuous column in simple construction using an RHS	Sheet 4 of 5	Rev

$$M_{x(2-1)} = \frac{3}{8} M_{2x} = 0.375 \times 28.0 = 10.5 \text{ kNm}$$

$$M_{y(2-1)} = \frac{3}{8} M_{2y} = 0.375 \times 1.75 = 0.66 \text{ kNm}$$

15.2.7 Combined axial force and moment check

Since only nominal moments are applied, the column should satisfy the relationship

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{p_y Z_y} \leq 1$$ 4.7.7

M_x = $M_{x(2-1)}$ = 10.5 kNm 4.3.6.1
M_y = $M_{y(2-1)}$ = 0.66 kNm

M_{bs} is the buckling resistance moment for simple columns. For a square hollow section, there is no need to consider lateral-torsional buckling, so M_{bs} equals M_c.

For a class 1 plastic section, M_c is given by:
$M_c = p_y S$ but $\leq 1.5\, p_y Z$
$p_y S = 355 \times 192 \times 10^{-3} = 68.2$ kNm
$1.5\, p_y Z = 1.5 \times 355 \times 163 \times 10^{-3} = 86.8$ kNm

Therefore, M_{bs} = 68.2 kNm.

$$\frac{640}{773} + \frac{10.5}{68.2} + \frac{0.66 \times 10^6}{355 \times 163 \times 10^3}$$

= 0.828 + 0.154 + 0.011 = 0.99 < 1
Therefore, **the combined resistance against axial force and moment is adequate.**

Adopt 150 x 150 x 6.3 HF RHS (28.1 kg/m) in S355

Example 15 Continuous column in simple construction using an RHS	Sheet 5 of 5	Rev

15.3 Blue Book approach

The values of P_c, M_{bs} and $p_y Z_y$ could have been obtained directly from Volume 1[2].

Try 150 x 150 x 6.3 HF RHS (28.1 kg/m) in S355

15.3.1 Compression resistance

For L_E = 5.0 m, P_c = 774 kN — Vol 1 Page D-149

F_c = 640 kN

640 kN < 774 kN

Therefore, **the compression resistance is adequate.**

15.3.2 Combined axial force and moment check

$p_y Z$ = 57.9 kNm — Vol 1 Page D-148

M_{bs} = M_c = 68.2 kNm

$$\frac{640}{774} + \frac{10.5}{68.2} + \frac{0.66}{57.9}$$

= 0.827 + 0.154 + 0.011 = 0.99 < 1

Therefore, **the combined resistance against axial force and moment is adequate.**

Note: Volume 1 only provides values for the terms in the interaction formula. The interaction check itself must still be carried out by hand.

Adopt 150 x 150 x 6.3 HF RHS (28.1 kg/m) in S355

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CALCULATION SHEET

Job No. CDS 153		Sheet 1 of 10	Rev
Job Title Example no. 16			
Subject Beam subject to combined biaxial bending and compression (compact section)			
Client SCI	Made by MDH	Date Jun 2003	
	Checked by ASM	Date Oct 2003	

16 Beam subject to combined biaxial bending and compression (compact section)

16.1 Introduction

The beam shown in Figure 16.1 is subject to axial compression applied at its ends, a vertical point load at mid-span and a horizontal uniformly distributed load out of plane. The beam is restrained against lateral movement and torsion at its mid-span by a secondary beam, but is otherwise unrestrained. It is assumed that the beam is pinned at its ends in the major and minor axis directions. Design the beam in S355 steel for the loading shown below. Two solutions are presented; the full calculation and the Blue Book approach.

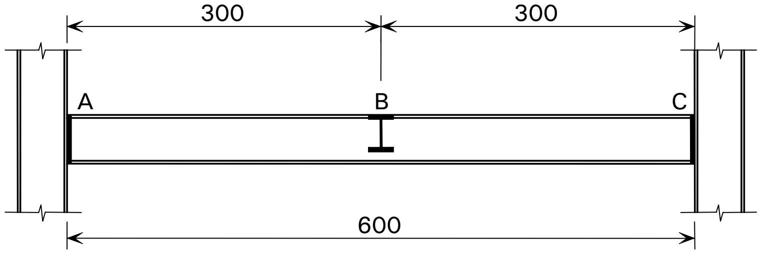

Figure 16.1

16.1.1 Loading (factored)

Factored compression force	F_c = 300 kN
Major axis point load	W = 100 kN
Major axis distributed load	w = 2 kN/m (assumed self-weight)
Minor axis distributed load	w = 3 kN/m

16.1.2 Bending moment and shear force

Major axis

The shear force and bending moment diagrams for major axis bending are as shown in Figure 16.2.

Maximum moment occurs at the centre:

$$M = \frac{wL^2}{8} + \frac{WL}{4} = \frac{2.0 \times 6.0^2}{8} + \frac{100 \times 6.0}{4} \quad = 159 \text{ kNm}$$

Shear force at the ends:

$$F_{ve} = \frac{wL}{2} + \frac{W}{2} = \frac{2.0 \times 6.0}{2} + \frac{100}{2} \quad = 56 \text{ kN}$$

Shear force at the centre:

$$F_{vc} = F_{ve} - \frac{wL}{2} \quad = 50 \text{ kN}$$

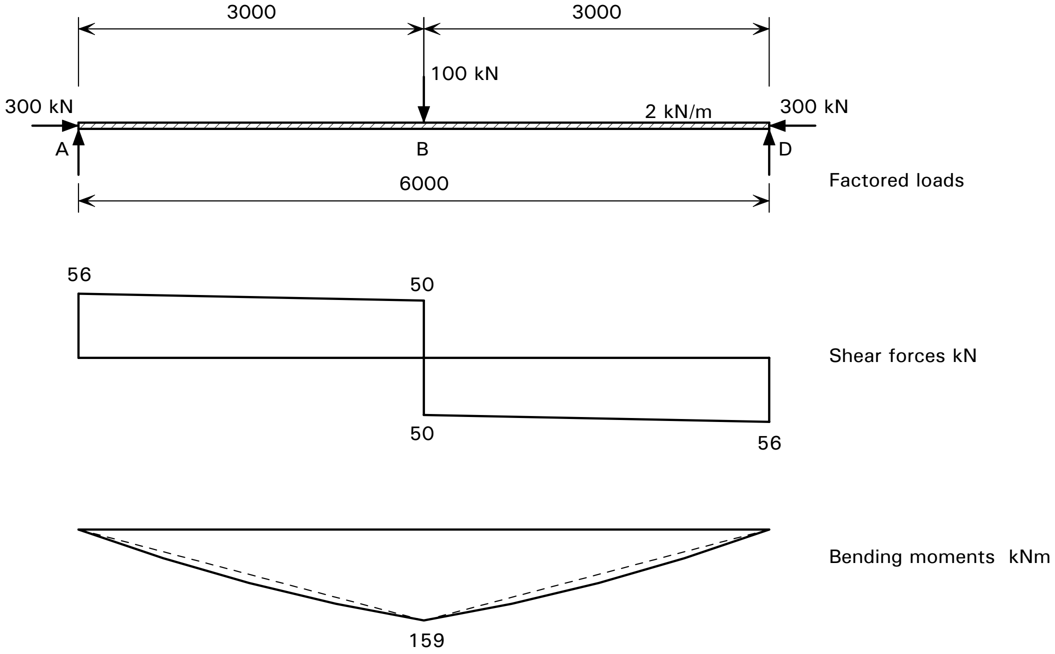

Figure 16.2 *Major axis bending*

Minor axis

The shear force and bending moment diagrams for minor axis bending are as shown in Figure 16.3.

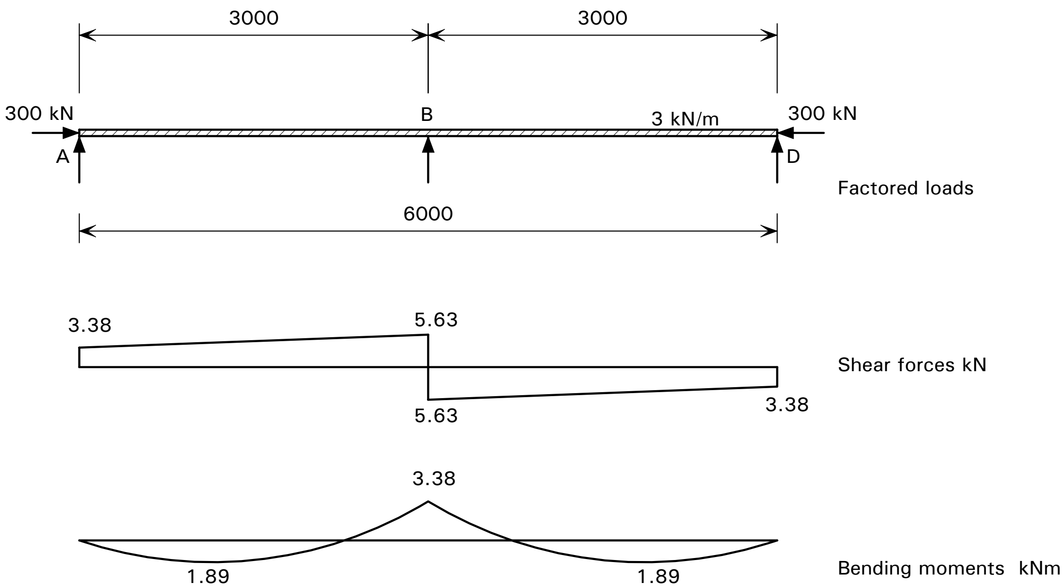

Figure 16.3 *Minor axis bending*

Maximum moment occurs at the central support:

$$M = \frac{wL^2}{8} = \frac{3.0 \times 3.0^2}{8} = 3.38 \text{ kNm}$$

From the shear force diagram, the maximum shear force also occurs at the central support.

$F_v = 5.63$ kN

16.2 Member checks

16.2.1 Trial section

Try 406 x 140 x 46 UB in grade S355

From section property tables:

Depth	D = 403.2 mm	Vol 1 Page B-4
Width	B = 142.2 mm	
Web thickness	t = 6.8 mm	
Flange thickness	T = 11.2 mm	
Depth between fillets	d = 360.4 mm	
Area of cross-section	A_g = 58.6 cm^2	
Plastic modulus	S_x = 888 cm^3	Vol 1 Page B-5
Plastic modulus	S_y = 118 cm^3	
Elastic modulus	Z_x = 778 cm^3	
Elastic modulus	Z_y = 75.7 cm^3	
Radius of gyration	r_x = 16.4 cm	
Radius of gyration	r_y = 3.03 cm	
Buckling parameter	u = 0.872	
Torsional index	x = 39.0	

Local buckling ratios: — Vol 1 Page B-4

Flange b/T = 6.35

Web d/t = 53.0

16.2.2 Section classification

Grade of steel = S355 — 3.1.1 Table 9

T < 16 mm

Therefore p_y = 355 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{355}} = 0.88$$

For the outstand element of a compression flange, the limiting b/T for a class 1 plastic flange is 9ε. Limiting b/T = 9×0.88 = 7.92 — 3.5.2 Table 11

The actual b/T = 6.35 < 7.92

Therefore, **the flange is class 1 plastic**.

For the web of an I- or H-section under axial compression and bending (the "generally" case in Table 11), the limiting d/t for a class 1 plastic web is — 3.5.2 Table 11

$$\frac{80\varepsilon}{1 + r_1} \text{ but} \geq 40\varepsilon$$

3.5.5

$$r_1 = \frac{F_c}{dtp_y} = \frac{300 \times 10^3}{360.4 \times 6.8 \times 355} = 34, \text{ but } -1 < r_1 \leq 1, \text{ therefore } r_1 = 0.34$$

$$\text{Limiting } d/t = \frac{80 \times 0.88}{1 + 0.34} = 52.5$$

The actual d/t = 53.0 > 52.5

Therefore, **the web is not class 1 plastic.**

Example 16 Biaxial bending and compression (compact section)	Sheet 4 of 10	Rev

The limiting d/t for a class 2 compact web is $\dfrac{100\varepsilon}{1+1.5r_1}$ but $\geq 40\varepsilon$ — 3.5.2 Table 11

Limiting d/t $= \dfrac{100 \times 0.88}{1+(1.5 \times 0.34)} = 58.3$

The actual d/t $= 53.0 < 58.3$

Therefore, **the web is class 2 compact.**

Since the flange is class 1 plastic and the web is class 2 compact**, the cross section is class 2 compact**.

16.2.3 Determine the effective length

The member is pinned at both ends about both axes, with an additional restraint against lateral (minor axis) deflection at its mid-span.

L_{Ex} = 6000 mm

L_{Ey} = 3000 mm

16.2.4 Slenderness

4.7.2

$$\lambda_x = \frac{L_{Ex}}{r_x} = \frac{6000}{164} = 36.6$$

$$\lambda_y = \frac{L_{Ey}}{r_y} = \frac{3000}{30.3} = 99.0$$

16.2.5 Check the compression resistance

Basic requirement $F_c \leq P_c$

$P_c = A_g p_c$ (for a compact cross section) — 4.7.4

The compressive strength p_c is obtained from the relevant strut curve for buckling about the x-x and y-y axes.

Buckling about the x-x axis

Use strut curve a — Table 23

For $\lambda_x = 36.6$ and $p_y = 355$ N/mm², $p_{cx} = 336$ N/mm² — Table 24a

$P_{cx} = 5860 \times 336 \times 10^{-3} = 1969$ kN

Buckling about the y-y axis

Use strut curve b — Table 23

For $\lambda_y = 99.0$ and $p_y = 355$ N/mm², $p_{cy} = 157$ N/mm² — Table 24b

$P_{cy} = 5860 \times 157 \times 10^{-3} = 920$ kN

Therefore, $P_c = 920$ kN

$F_c = 300$ kN

300 kN < 920 kN

Therefore, **the compression resistance is adequate.**

16.2.6 Check for shear buckling

If the d/t ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000[1]. — 4.2.3

$70\varepsilon = 70 \times 0.88 = 61.6$

$d/t = 53.0 < 61.6$, so **there is no need to check for shear buckling**.

16.2.7 Check the shear capacity

Major axis

Basic requirement $F_v \leq P_v$ 4.2.3

$P_v = 0.6\, p_y A_v$

$p_y = 355\ \text{N/mm}^2$

$A_v = tD = 6.8\ \ 403.2 = 2742\ \text{mm}^2$

$P_v = 0.6 \times 355 \times 2742 \times 10^{-3} = 584\ \text{kN}$

Maximum shear force $F_v = 56$ kN.

56 kN < 584 kN

Therefore, **the major axis shear capacity is adequate.**

Minor axis

Basic requirement $F_v \leq P_v$ 4.2.3

$P_v = 0.6\, p_y A_v$

$p_y = 355\ \text{N/mm}^2$

$A_v = 0.9\, A_o$, where A_o is the combined area of the two flanges.

$A_v = 0.9 \times 2 \times 142.2 \times 11.2 = 2867\ \text{mm}^2$

$P_v = 0.6 \times 355 \times 2867 \times 10^{-3} = 611\ \text{kN}$

Maximum shear force $F_v = 5.63$ kN.

5.63 kN < 611 kN

Therefore, **the minor axis shear capacity is adequate.**

16.2.8 Check the moment capacity

Major axis

Basic requirement $M_x \leq M_{cx}$

Check whether the shear is "high" (i.e. $F_v > 0.6\ P_v$) or "low" at the point of maximum moment. 4.2.5.2

At the centre of the member, $F_{vc} = 50$ kN

$0.6\ P_v = 0.6 \times 584 = 350$ kN

50 kN < 350 kN

Therefore, **the shear is low**.

For low shear, the moment capacity for a class 2 section is given by: 4.2.5.2

$M_{cx} = p_y S_x$

$M_{cx} = 355 \times 888 \times 10^{-3} = 315$ kNm

Check limit to avoid irreversible deformation under serviceability loads. 4.2.5.1

For a simply supported beam $M_{cx} \leq 1.2\ p_y Z_x$

$1.2 p_y Z_x = 1.2 \times 355 \times 778 \times 10^{-3} = 331$ kNm

Therefore $M_{cx} = 315$ kNm

From Figure 16.2, $M_x = 59$ kNm

159 kNm < 315 kNm

Therefore, **the major axis moment capacity is adequate.**

Minor axis

Basic requirement $M_y \leq M_{cy}$

By inspection, the shear is low (i.e. $F_v < 0.6\ P_v$). 4.2.5.2

For low shear, the moment capacity for a class 2 section is given by

$M_{cy} = p_y S_y$

$M_{cy} = 355 \times 118 \times 10^{-3} = 41.9$ kNm

Example 16 Biaxial bending and compression (compact section)	Sheet 6 of 10	Rev

Check limit to avoid irreversible deformation under serviceability loads. *4.5.2.1*
For a continuous beam $M_{cy} \leq 1.5\, p_y Z_y$
$1.5 p_y Z_y = 1.5 \times 355 \times 75.7 \times 10^{-3} = 40.3$ kNm

Therefore $M_{cy} = 40.3$ kNm
From Figure 16.3, $M_y = 3.38$ kNm
3.38 kNm < 40.3 kNm
Therefore, **the minor axis moment capacity is adequate.**

16.2.9 Lateral-torsional buckling

The beam is restrained against lateral-torsional buckling at its supports and at its mid-span. There are, therefore, two segments to consider. However, due to symmetry, it is sufficient to design one segment only in this example. Consider segment length AB.

Basic requirements: $M_x \leq M_b / m_{LT}$ *4.3.6.2*
and $M_x \leq M_{cx}$ (already checked in 16.2.8 above)

The buckling resistance moment M_b for a class 2 compact section is given by: *4.3.6.4*

$M_b = p_b S_x$

where p_b is the bending strength and is dependent on the design strength p_y and the equivalent slenderness λ_{LT}.

4.3.6.7

$$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$$

$$\lambda/x = \frac{99}{39.0} = 2.5$$

For a section with equal flanges and $\lambda/x = 2.5$, *Table 19*
$v = 0.93$

For a class 2 compact section, $\beta_W = 1.0$ *4.3.6.9*
Therefore, $\lambda_{LT} = 0.872 \times 0.93 \times 99.0 = 80.3$
For $p_y = 355$ N/mm² and $\lambda_{LT} = 80.3$, Table 16 gives $p_b = 189$ N/mm² *Table 16*
$M_b = 189 \times 888 \times 10^{-3} = 168$ kNm

Since there is no major axis loading between the restraints (apart from the self-weight of the beam, which is considered insignificant for the purpose of evaluating m_{LT}), m_{LT} can be obtained directly from Table 18 for a known value of β.
β is the ratio of the bending moments at points A and B, i.e. $\beta = 0$ *4.3.6.6*
From Table 18, $m_{LT} = 0.6$ *Table 18*

$M_b / m_{LT} = 168 / 0.6 = 280$ kNm

From Figure 16.2, $M_x = 159$ kNm
159 kNm < 280 kNm
Therefore, **the buckling resistance moment is adequate.**

Example 16 Biaxial bending and compression (compact section)	Sheet 7 of 10	Rev

16.2.10 Interaction between axial load and bending

Cross section capacity

Basic requirement: $\dfrac{F_c}{A_g p_y} + \dfrac{M_x}{M_{cx}} + \dfrac{M_y}{M_{cy}} \le 1$ 4.8.3.2

At point B

$$\frac{300 \times 10^3}{58.6 \times 10^2 \times 355} + \frac{159}{315} + \frac{3.38}{40.3}$$

$= 0.144 + 0.505 + 0.084 \quad = 0.73 < 1$

Therefore, **the cross section capacity is adequate.**

Member buckling resistance

BS 5950-1:2000 presents two methods for checking the member buckling resistance: the simplified method (Clause 4.8.3.3.1) and the more exact method (Clause 4.8.3.3.2). In the simplified method, the following relationships must be satisfied:

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \le 1 \quad \text{and}$$ 4.8.3.3.1

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \le 1$$

M_{LT} is the maximum major axis moment in segment AB $= 159$ kNm. 4.8.3.3.1

m_x is determined between restraints on the x axis, i.e. A and C, according to the shape of the bending moment diagram (see Figure 16.2).

As this is one of the specific cases listed in BS 5950-1:2000[1], the value of m_x can be obtained directly from Table 26.

$m_x = 0.9$ 4.8.3.3.4 Table 26

m_y is determined between restraints on the y axis, i.e. A and B or B and C, according to the shape of the bending moment diagram between those restraints as shown in Figure 16.4.

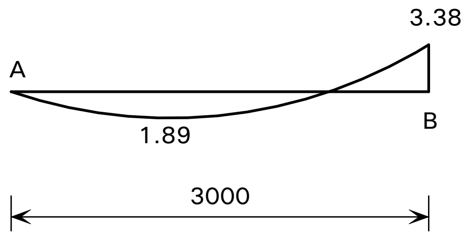

Figure 16.4 *Minor axis moment*

Example 16 Biaxial bending and compression (compact section)	Sheet 8 of 10	Rev

In this case, m_y must be obtained from the following general formula.

4.8.3.3.4
Table 26

$$m_y = 0.2 + \frac{0.1\,M_2 + 0.6\,M_3 + 0.1\,M_4}{M_{max}} \quad \text{but } m_y \geq \frac{0.8\,M_{24}}{M_{max}}$$

The moments M_2 and M_4 are the values at the quarter points and the moment M_3 is the value at mid-length. M_{max} is the maximum moment in the segment and M_{24} is the maximum moment in the central half of the segment.

Location	M_2	M_3	M_4	M_{24}	M_{max}
Moment kNm	1.69	1.69	0	1.89	3.38

$$m_y = 0.2 + \frac{0.1 \times 1.69 + 0.6 \times 1.69 + 0}{3.38} = 0.55 \quad \text{but } m_y \geq \frac{0.8 \times 1.89}{3.38} = 0.45$$

Therefore, $m_y = 0.55$

$$\frac{300}{920} + \frac{0.9 \times 159 \times 10^6}{355 \times 778 \times 10^3} + \frac{0.55 \times 3.38 \times 10^6}{355 \times 75.7 \times 10^3}$$

$$= 0.326 + 0.518 + 0.069 = 0.91 < 1$$

$$\frac{300}{920} + \frac{0.6 \times 159}{168} + \frac{0.55 \times 3.38 \times 10^6}{355 \times 75.7 \times 10^3}$$

$$= 0.326 + 0.568 + 0.069 = 0.96 < 1$$

Therefore, **the member buckling resistance is adequate.**

Adopt 406 x 140 x 46 UB in S355

Note: 1. If appropriate, the web should be checked for bearing and buckling at the supports and at the point of load application, as in Example 2.

2. The deflections should be checked at the serviceability limit state in accordance with the recommendations in clause 2.5.2.

Example 16 Biaxial bending and compression (compact section)	Sheet 9 of 10	Rev

16.3 Blue Book approach

The section capacities and member resistances calculated in section 16.2 could have been obtained directly from Volume 1[2].

Try 406 x 140 x 46 UB in grade S355

16.3.1 Compression resistance

For L_{Ex} = 6.0 m, P_{cx} = 1970 kN
For L_{Ey} = 3.0 m, P_{cy} = 925 kN
Therefore, P_c = 925 kN
F_c = 300 kN
300 kN < 925 kN
Therefore, **the compression resistance is adequate.**

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16.3.2 Shear Capacity (major axis)

P_v = 584 kN (no value given for minor axis shear)
F_v = 56 kN
56 kN < 584 kN
Therefore, **the major axis shear capacity is adequate.**

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16.3.3 Moment capacity

For F/P_z = 0.144
M_{cx} = 315 kNm
M_x = 159 kNm
159 kNm < 315 kNm
Therefore, **the major axis moment capacity is adequate.**

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For F/P_z = 0.144
M_{cy} = 32.2 kNm
M_y = 3.38 kNm
3.38 kNm < 32.2 kNm
Therefore, **the minor axis moment capacity is adequate.**

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The value of M_{cy} given by Volume1 is considerably lower than that calculated in Section 16.2 above (40.3 kNm) due to the fact that Volume 1 uses the limit of 1.2 $p_y Z$, whereas the higher limit of 1.5$p_y Z$ is more appropriate in this example, since the beam is continuous over a central support for minor axis bending.

16.3.4 Buckling resistance moment

For L_E = 3.0 m and F/P_z = 0.144
M_b = 167 kNm

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For a quick approximation, m_{LT} may conservatively be taken as 1.0. However, more accurately, m_{LT} should be obtained from Table 18 of BS 5950-1:2000 as in Section 16.2.9 of this example.

m_{LT} = 0.6
M_b / m_{LT} = 167 / 0.6 = 278 kNm
M_x = 159 kNm
159 kNm < 278 kNm
Therefore, **the buckling resistance moment is adequate.**

Table 18

Example 16 Biaxial bending and compression (compact section)	Sheet 10 of 10	Rev

16.3.5 Interaction between axial load and bending

Cross section capacity at point B

$A_g p_y$ = 2080 kN (Vol 1 Page D-115)

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} = \frac{300}{2080} + \frac{159}{315} + \frac{3.38}{32.2}$$

$$= 0.144 + 0.505 + 0.105 \quad = 0.75 < 1$$

Therefore, **the cross section capacity is adequate.**

Member buckling resistance (4.8.3.3.4)

For a quick approximation, m_x and m_y may conservatively be taken as 1.0. However, more accurately, m_x and m_y should be obtained from Table 26 of BS 5950-1:2000 as in Section 16.2.10 of this example.

m_x = 0.9 (Table 26)

m_y = 0.55

$p_y Z_x$ = 276 kNm (Vol 1 Page D-115)

$p_y Z_y$ = 26.9 kNm

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} = \frac{300}{925} + \frac{0.9 \times 159}{276} + \frac{0.55 \times 3.38}{26.9}$$

$$= 0.324 + 0.518 + 0.069 = 0.91 < 1$$

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} = \frac{300}{925} + \frac{0.6 \times 159}{167} + \frac{0.55 \times 3.38}{26.9}$$

$$= 0.324 + 0.571 + 0.069 = 0.96 < 1$$

Therefore, **the member buckling resistance is adequate.**

Adopt 406 x 140 x 46 UB in S355

Note: 1. If appropriate, the web should be checked for bearing and buckling at the supports and at the point of load application, as in Example 2.

2. The deflections should be checked at the serviceability limit state in accordance with the recommendations in clause 2.5.2.

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CALCULATION SHEET

Job No.	CDS 153	Sheet 1 of 10	Rev
Job Title	Example no. 17		
Subject	Beam subject to combined biaxial bending and compression (semi-compact section)		
Client	SCI	Made by MDH	Date Jun 2003
		Checked by ASM	Date Oct 2003

17 Beam subject to combined biaxial bending and compression (semi-compact section)

17.1 Introduction

The beam considered in Example 16 is required to carry an increased compression force of 1200 kN. Redesign the beam in S355 steel for the new loading. Two solutions are presented; the full calculation and the Blue Book approach.

17.1.1 Loading (factored)

Factored compression force F_c = 1200 kN
Major axis point load W = 100 kN
Major axis distributed load w = 2 kN/m (assumed self-weight)
Minor axis distributed load w = 3 kN/m

17.1.2 Bending moment and shear force

Major axis

The shear force and bending moment diagrams for major axis bending are as shown in Figure 17.1.

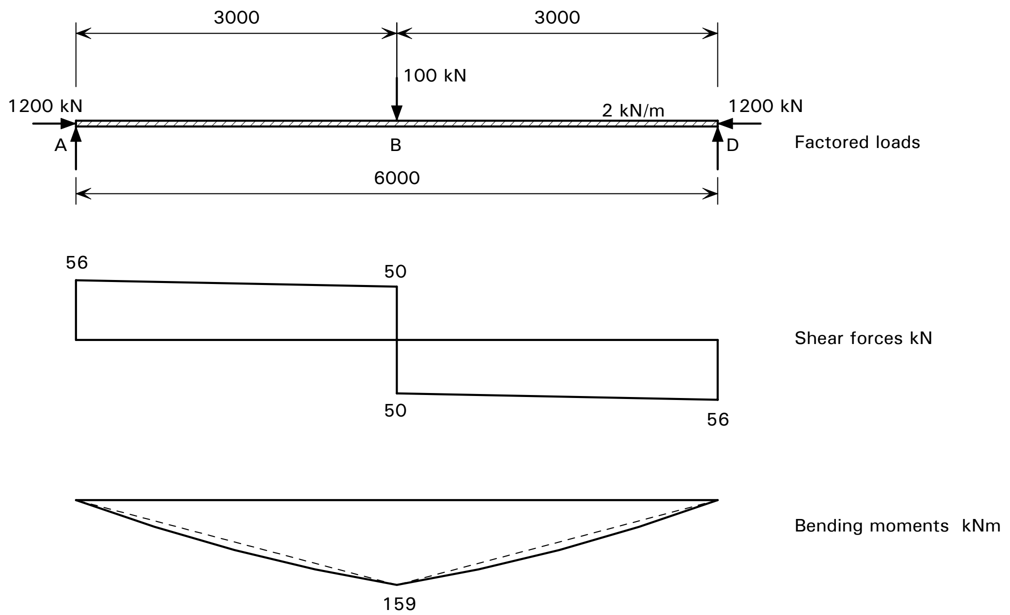

Figure 17.1 *Major axis bending*

Example 17 Biaxial bending and compression (semi-compact section)	Sheet 2 of 10	Rev

Maximum moment occurs at the centre:

$$M = \frac{wL^2}{8} + \frac{WL}{4} \quad = \frac{2.0 \times 6.0^2}{8} + \frac{100 \times 6.0}{4} \quad = 159 \text{ kNm}$$

Shear force at the ends:

$$F_{ve} = \frac{wL}{2} + \frac{W}{2} \quad = \frac{2.0 \times 6.0}{2} + \frac{100}{2} \quad = 56 \text{ kN}$$

Shear force at the centre:

$$F_{vc} = F_{ve} - \frac{wL}{2} \quad = 50 \text{ kN}$$

Minor axis

The shear force and bending moment diagrams for minor axis bending are as shown in Figure 17.2.

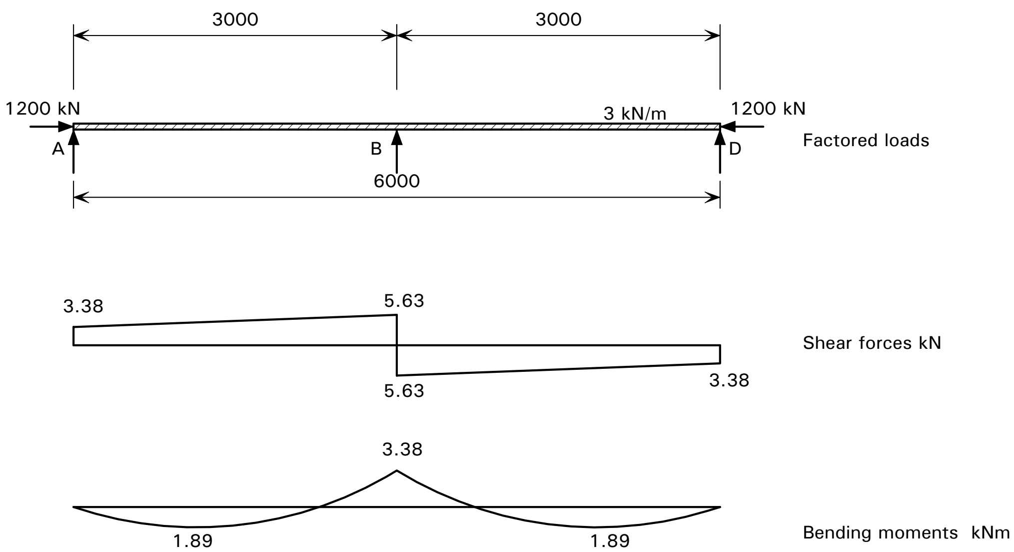

Figure 17.2 *Minor axis bending*

Maximum moment occurs at the central support:

$$M = \frac{wL^2}{8} \quad = \frac{3.0 \times 3.0^2}{8} \quad = 3.38 \text{ kNm}$$

From the shear force diagram, the maximum shear force also occurs at the central support.

$F_v = 5.63$ kN

17.2 Member checks

17.2.1 Trial section

Try 457 x 191 x 67 UB in grade S355

From section property tables:

Depth	D	= 453.4 mm	Vol 1 Page B-4
Width	B	= 189.9 mm	
Web thickness	t	= 8.5 mm	
Flange thickness	T	= 12.7 mm	
Depth between fillets	d	= 407.6 mm	
Area of cross-section	A_g	= 85.5 cm^2	Vol 1 Page B-5
Plastic modulus	S_x	= 1470 cm^3	
Plastic modulus	S_y	= 237 cm^3	
Elastic modulus	Z_x	= 1300 cm^3	
Elastic modulus	Z_y	= 153 cm^3	
Radius of gyration	r_x	= 18.5 cm	
Radius of gyration	r_y	= 4.12 cm	
Buckling parameter	u	= 0.872	
Torsional index	x	= 37.9	

Local buckling ratios: — Vol 1 Page B-4

Flange	b/T	= 7.48	
Web	d/t	= 48.0	

17.2.2 Section classification

Grade of steel = S355 — 3.1.1 Table 9

$T < 16$ mm

Therefore p_y = 355 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{355}} = 0.88$$

For the outstand element of a compression flange, the limiting b/T for a class 1 plastic flange is 9ε. Limiting $b/T = 9 \times 0.88 = 7.92$ — 3.5.2 Table 11

The actual $b/T = 7.48 < 7.92$

Therefore, **the flange is class 1 plastic**.

For the web of an I- or H-section under axial compression and bending (the "generally" case in Table 11), the limiting d/t for a class 1 plastic web is — 3.5.2 Table 11

$$\frac{80\varepsilon}{1 + r_1} \text{ but } \geq 40\varepsilon$$

$$r_1 = \frac{F_c}{dtp_y} = \frac{1200 \times 10^3}{407.6 \times 8.5 \times 355} = 0.98\text{, but } -1 < r_1 \leq 1\text{, therefore } r_1 = 0.98$$ — 3.5.5

$$\text{Limiting } d/t = \frac{80 \times 0.88}{1 + 0.98} = 35.6 \text{ but } \geq 40 \times 0.88 = 35.2$$

The actual $d/t = 48.0 > 35.6$

Therefore, **the web is not class 1 plastic.**

Example 17 Biaxial bending and compression (semi-compact section)	Sheet 4 of 10	Rev

The limiting *d/t* for a class 2 compact web is $\dfrac{100\varepsilon}{1+1.5r_1}$ but $\geq 40\varepsilon$	3.5.2 Table 11
Limiting *d/t* $= \dfrac{100 \times 0.88}{1+(1.5 \times 0.98)} = 35.6$ but $\geq 40 \times 0.88 = 35.2$	
The actual *d/t* $= 48.0 > 35.6$ Therefore, **the web is not class 2 compact.**	
The limiting *d/t* for a class 3 semi-compact web is $\dfrac{120\varepsilon}{1+2r_2}$ but $\geq 40\varepsilon$	3.5.2 Table 11
$r_2 = \dfrac{F_c}{A_g p_y} = \dfrac{1200 \times 10^3}{8550 \times 355} = 40$	
Limiting *d/t* $= \dfrac{120 \times 0.88}{1+(2 \times 0.40)} = 58.7$ but $\geq 40 \times 0.88 = 35.2$	
The actual *d/t* $= 48.0 < 58.7$ Therefore, **the web is class 3 semi-compact.**	
Since the flange is class 1 plastic and the web is class 3 semi-compact**, the cross section is class 3 semi-compact**.	
17.2.3 Determine the effective length	
The member is pinned at both ends about both axes, with an additional restraint against lateral (minor axis) deflection at its mid-span.	
$L_{Ex} = 6000$ mm $L_{Ey} = 3000$ mm	
17.2.4 Slenderness	
$\lambda_x = \dfrac{L_{Ex}}{r_x} = \dfrac{6000}{185} = 32.4$	4.7.2
$\lambda_y = \dfrac{L_{Ey}}{r_y} = \dfrac{3000}{41.2} = 72.8$	
17.2.5 Check the compression resistance	
Basic requirement $F_c \leq P_c$ $P_c = A_g p_c$	4.7.4
The compressive strength p_c is obtained from the relevant strut curve for buckling about the x-x and y-y axes.	
Buckling about the x-x axis	
Use strut curve a For $\lambda_x = 32.4$ and $p_y = 355$ N/mm², $p_{cx} = 341$ N/mm² $P_{cx} = 8550 \times 341 \times 10^{-3} = 2916$ kN	Table 23 Table 24a

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Buckling about the y-y axis

Use strut curve b — Table 23

For $\lambda_y = 72.8$ and $p_y = 355$ N/mm^2, $p_{cy} = 234$ N/mm^2 — Table 24b

$P_{cy} = 8550 \times 234 \times 10^{-3} = 2001$ kN

$P_c = \min(P_{cx}, P_{cy}) = 2001$ kN

$F_c = 1200$ kN

1200 kN < 2001 kN

Therefore, **the compression resistance is adequate.**

17.2.6 Check for shear buckling

If the d/t ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000 [1]. — 4.2.3

$70\varepsilon = 70 \times 0.88 = 61.6$

$d/t = 48.0 < 61.6$, so **there is no need to check for shear buckling.**

17.2.7 Check the shear capacity

Major axis

Basic requirement $F_v \leq P_v$ — 4.2.3

$P_v = 0.6\, p_y A_v$

$p_y = 355$ N/mm^2

$A_v = tD = 8.5 \times 453.4 = 3854$ mm^2

$P_v = 0.6 \times 355 \times 3854 \times 10^{-3} = 821$ kN

Maximum shear force $F_v = 56$ kN.

56 kN < 821 kN

Therefore, **the major axis shear capacity is adequate.**

Minor axis

Basic requirement $F_v \leq P_v$ — 4.2.3

$P_v = 0.6\, p_y A_v$

$p_y = 355$ N/mm^2

$A_v = 0.9\, A_o$, where A_o is the combined area of the two flanges

$A_v = 0.9 \times 2 \times 189.9 \times 12.7 = 4341$ mm^2

$P_v = 0.6 \times 355 \times 4341 \times 10^{-3} = 925$ kN

Maximum shear force $F_v = 5.63$ kN

5.63 kN < 925 kN

Therefore, **the minor axis shear capacity is adequate.**

17.2.8 Check the moment capacity

Major axis

Basic requirement $M_x \leq M_{cx}$

Check whether the shear is "high" (i.e. $F_v > 0.6\, P_v$) or "low" at the point of maximum moment.

At the centre of the member, $F_{vc} = 50$ kN

$0.6\, P_v = 0.6 \times 821 = 493$ kN

50 kN < 493 kN

Therefore, **the shear is low.**

For low shear, the moment capacity for a class 3 section is given by

$M_{cx} = p_y Z_x$ or alternatively $M_{cx} = p_y S_{x,\mathrm{eff}}$

Example 17 Biaxial bending and compression (semi-compact section)	Sheet 6 of 10	Rev

$$S_{\mathrm{x,eff}} = Z_{\mathrm{x}} + (S_{\mathrm{x}} - Z_{\mathrm{x}})\left[\frac{\left(\frac{\beta_{3\mathrm{w}}}{d/t}\right)^2 - 1}{\left(\frac{\beta_{3\mathrm{w}}}{\beta_{2\mathrm{w}}}\right)^2 - 1}\right] \text{ but } S_{\mathrm{x,eff}} \le Z_{\mathrm{x}} + (S_{\mathrm{x}} - Z_{\mathrm{x}})\left[\frac{\frac{\beta_{3\mathrm{f}}}{b/T} - 1}{\frac{\beta_{3\mathrm{f}}}{\beta_{2\mathrm{f}}} - 1}\right]$$ 3.5.6.2

The terms are the limiting d/t and b/T ratios obtained from Table 11. The number in the subscript refers to the classification and the w and f refer to web and flange respectively.

$\beta_{2\mathrm{w}}$ = 35.6, i.e the limiting d/t for a class 2 compact web (Sheet 3 of this example).

$\beta_{3\mathrm{w}}$ = 58.7, i.e. the limiting d/t for a class 3 compact web (Sheet 4 of this example)

$$S_{\mathrm{x,eff}} = 1300 + (1470 - 1300)\left[\frac{\left(\frac{58.7}{48.0}\right)^2 - 1}{\left(\frac{58.7}{35.6}\right)^2 - 1}\right] = 1349 \text{ cm}^3$$

$$S_{\mathrm{x,eff}} \le 1300 + (1470 - 1300)\left[\frac{\frac{13.2}{7.48} - 1}{\frac{13.2}{8.8} - 1}\right] = 1560 \text{ cm}^3$$

In this case, the first of the two values of $S_{\mathrm{x,eff}}$ is the lower and should be used. This is usually the case for a UB, since the moment capacity is limited by local buckling in the web. The second equation, which relates to the flanges, yields a value of $S_{\mathrm{x,eff}}$ greater than S_{x}. This value is meaningless since $S_{\mathrm{x,eff}}$ must lie between Z_{x} and S_{x} and is due to the fact that, in this case, the flanges are class 1 plastic.

M_{cx} = 355 × 1349 × 10^{-3} = 479 kNm

Check limit to avoid irreversible deformation under serviceability loads.
For a simply supported beam $M_{\mathrm{cx}} \le 1.2\, p_{\mathrm{y}} Z_{\mathrm{x}}$
$1.2 p_{\mathrm{y}} Z_{\mathrm{x}}$ = 1.2 × 355 × 1300 × 10^{-3} = 554 kNm 4.2.5.1

Therefore M_{cx} = 479 kNm
From Figure 17.1, M_x = 159 kNm
159 kNm < 479 kNm
Therefore, **the major axis moment capacity is adequate.**

Minor axis

Basic requirement $M_{\mathrm{y}} \le M_{\mathrm{cy}}$ 4.2.5.2
By inspection, the shear is low (i.e. $F_{\mathrm{v}} < 0.6\, P_{\mathrm{v}}$)
For low shear, the moment capacity for a class 3 section is given by:
$M_{\mathrm{cy}} = p_{\mathrm{y}} Z_{\mathrm{y}}$ or alternatively $M_{\mathrm{cy}} = p_{\mathrm{y}} S_{\mathrm{y,eff}}$

$$S_{\mathrm{y,eff}} = Z_{\mathrm{y}} + (S_{\mathrm{y}} - Z_{\mathrm{y}})\left[\frac{\frac{\beta_{3\mathrm{f}}}{b/T} - 1}{\frac{\beta_{3\mathrm{f}}}{\beta_{2\mathrm{f}}} - 1}\right] = 153 + (237 - 153)\left[\frac{\frac{13.2}{7.48} - 1}{\frac{13.2}{8.8} - 1}\right] = 281 \text{ cm}^3$$ 3.5.6.2

Example 17 Biaxial bending and compression (semi-compact section)	Sheet 7 of 10	Rev

This value of $S_{y,eff}$ is greater than S_y and should not be used. As with the second formula for $S_{x,eff}$ the unrealistic value is due to the fact that the flanges are class 1, while these formulae are only designed to work with class 3 elements. In this case, the full plastic modulus S_y should be used in place of the calculated $S_{y,eff}$. This is acceptable since local buckling in the web is unlikely to reduce the minor axis capacity of the section (the web lies along the neutral axis).

$M_{cy} = 355 \times 237 \times 10^{-3} = 84.1$ kNm

Check limit to avoid irreversible deformation under serviceability loads.

For a continuous beam $M_{cx} \leq 1.5\, p_y Z_y$ — 4.5.2.1

$1.5 p_y Z_y = 1.5 \times 355 \times 153 \times 10^{-3} = 81.5$ kNm

Therefore $M_{cy} = 81.5$ kNm

From Figure 17.2, $M_y = 3.38$ kNm

3.38 kNm < 81.5 kNm

Therefore, **the minor axis moment capacity is adequate.**

17.2.9 Lateral-torsional buckling

The beam is restrained against lateral-torsional buckling at its supports and at its mid-span. There are, therefore, two segments to consider. However, due to symmetry, it is sufficient to design one segment only in this example. Consider segment length AB.

Basic requirements: $M_x \leq M_b / m_{LT}$

and $M_x \leq M_{cx}$ (already checked in 17.2.8 above) — 4.3.6.2

The buckling resistance moment M_b for a class 3 semi-compact section is given by:

$M_b = p_b S_{x,eff}$ — 4.3.6.4

where p_b is the bending strength and is dependent on the design strength p_y and the equivalent slenderness λ_{LT}

$$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$$ — 4.3.6.7

$$\lambda/x = \frac{72.8}{37.9} = 1.9$$

For a section with equal flanges and $\lambda/x = 1.9$, $v = 0.96$ — Table 19

$$\beta_w = \frac{S_{x,eff}}{S_x} = \frac{1349}{1470} = 0.918 \qquad \sqrt{\beta_w} = 0.958$$ — 4.3.6.9

Therefore, $\lambda_{LT} = 0.872 \times 0.96 \times 72.8 \times 0.958 = 58.4$

For $p_y = 355$ N/mm² and $\lambda_{LT} = 58.4$, Table 16 gives $p_b = 262$ N/mm² — Table 16

$M_b = 262 \times 1349 \times 10^{-3} = 353$ kNm

Since there is no major axis loading between the restraints (apart from the self-weight of the beam, which is considered insignificant for the purpose of evaluating m_{LT}), m_{LT} can be obtained directly from Table 18 for a known value of β.

Example 17 Biaxial bending and compression (semi-compact section)	Sheet 8 of 10	Rev

β is the ratio of the bending moments at points A and B, i.e. $\beta = 0$ — 4.3.6.6

From Table 18, $m_{LT} = 0.6$ — Table 18

$M_b / m_{LT} = 353 / 0.6 = 588$ kNm

From Figure 17.2, $M_x = 159$ kNm

159 kNm < 588 kNm

Therefore, **the buckling resistance moment is adequate.**

17.2.10 Interaction between axial load and bending

Cross section capacity

Basic requirement: $$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

At point B $$\frac{1200 \times 10^3}{85.5 \times 10^2 \times 355} + \frac{159}{479} + \frac{3.38}{81.5}$$

$$= 0.395 + 0.332 + 0.041 \quad = 0.77 < 1$$

Therefore, **the cross section capacity is adequate.**

Member buckling resistance

BS 5950-1:2000 presents two methods for checking the member buckling resistance: the simplified method (Clause 4.8.3.3.1) and the more exact method (Clause 4.8.3.3.2). In the simplified method, the following relationships must be satisfied:

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \leq 1 \quad \text{and}$$ 4.8.3.3.1

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \leq 1$$ 4.8.3.3.1

M_{LT} is the maximum major axis moment in segment AB $= 159$ kNm — 4.8.3.3.4

$m_x = 0.9$ and $m_y = 0.55$ (see Example 16) — Table 26

$$\frac{1200}{2001} + \frac{0.9 \times 159 \times 10^6}{355 \times 1300 \times 10^3} + \frac{0.55 \times 3.38 \times 10^6}{355 \times 153 \times 10^3}$$

$$= 0.600 + 0.310 + 0.034 \quad = 0.94 < 1$$

$$\frac{1200}{2001} + \frac{0.6 \times 159}{353} + \frac{0.55 \times 3.38 \times 10^6}{355 \times 153 \times 10^3}$$

$$0.600 + 0.270 + 0.034 \quad = 0.90 < 1$$

Therefore, **the member buckling resistance is adequate.**

<u>Adopt 457 x 191 x 67 UB in S355</u>

Note: 1. If appropriate, the web should be checked for bearing and buckling at the supports and at the point of load application, as in Example 2.

2. The deflections should be checked at the serviceability limit state in accordance with the recommendations in clause 2.5.2.

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17.3 Blue Book approach

The section capacities and member resistances calculated in section 17.2 above could have been obtained directly from Volume 1[2].

Try 457 x 191 x 67 UB in grade S355

17.3.1 Compression resistance

For L_{Ex} = 6.0 m, P_{cx} = 2910 kN
For L_{Ey} = 3.0 m, P_{cy} = 2000 kN
Therefore, P_c = 2000 kN
F_c = 1200 kN
1200 kN < 2000 kN
Therefore, **the compression resistance is adequate.**

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17.3.2 Shear Capacity (major axis)

P_v = 821 kN (no value given for minor axis shear)
F_v = 56 kN
56 kN < 821 kN
Therefore, **the major axis shear capacity is adequate.**

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17.3.3 Moment capacity

For F/P_z = 0.4
M_{cx} = 478 kNm
M_x = 159 kNm
159 kNm < 478 kNm
Therefore, **the major axis moment capacity is adequate.**

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Page D-110

For F/P_z = 0.4
M_{cy} = 65.2 kNm
M_y = 3.38 kNm
3.38 kNm < 65.2 kNm
Therefore, **the minor axis moment capacity is adequate.**

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As in Example 16, the difference between the value of M_{cy} given by Volume 1 and that calculated in Section 17.2 above is due to the fact that Volume 1 uses the limit of 1.2 $p_y Z$, whereas this example uses $1.5 p_y Z$. The use of this higher limit is more appropriate, since the beam is continuous over a central support for minor axis bending.

17.3.4 Buckling resistance moment

For L_E = 3.0 m and F/P_z = 0.4
M_b = 346 kNm

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For a quick approximation, m_{LT} may conservatively be taken as 1.0. However, more accurately, m_{LT} should be obtained from Table 18 of BS 5950-1:2000 as in Section 17.2.9 of this example.

m_{LT} = 0.6
M_b / m_{LT} = 346 / 0.6 = 577 kNm
M_x = 159 kNm
159 kNm < 577 kNm
Therefore, **the buckling resistance moment is adequate.**

Table 18

Example 17 Biaxial bending and compression (semi-compact section)	Sheet 10 of 10	Rev

17.3.5 Interaction between axial load and bending

Cross section capacity at point B

$A_g p_y$ = 3040 kN

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} = \frac{1200}{3040} + \frac{159}{478} + \frac{3.38}{65.2}$$

= 0.395 + 0.333 + 0.052 = 0.78 < 1

Therefore, **the cross section capacity is adequate.**

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Member buckling resistance

4.8.3.3.4

For a quick approximation, m_x and m_y may conservatively be taken as 1.0. However, more accurately, m_x and m_y should be obtained from Table 26 of BS 5950-1:2000 as in Section 17.2.10 of this example.

m_x = 0.9
m_y = 0.55

Table 26

$p_y Z_x$ = 460 kNm
$p_y Z_y$ = 54.3 kNm

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$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} = \frac{1200}{2000} + \frac{0.9 \times 159}{460} + \frac{0.55 \times 3.38}{54.3}$$

= 0.600 + 0.311 + 0.034 = 0.95 < 1

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} = \frac{1200}{2000} + \frac{0.6 \times 159}{346} + \frac{0.55 \times 3.38}{54.3}$$

= 0.600 + 0.276 + 0.034 = 0.91 < 1

Therefore, **the member buckling resistance is adequate.**

Adopt 457 x 191 x 67 UB in S355

Note: 1. If appropriate, the web should be checked for bearing and buckling at the supports and at the point of load application, as in Example 2.

2. The deflections should be checked at the serviceability limit state in accordance with the recommendations in clause 2.5.2.

The Steel Construction Institute	Job No.	CDS 153		Sheet 1 of 9	Rev
Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 623345 Fax: (01344) 622944	Job Title	Example no. 18			
	Subject	Beam subject to combined biaxial bending and compression (slender section)			
CALCULATION SHEET	Client SCI	Made by	MDH	Date	Jun 2003
		Checked by	ASM	Date	Oct 2003

18 Beam subject to combined biaxial bending and compression (slender section)

18.1 Introduction

The beam considered in Example 16 is required to carry an increased compression force of 2400 kN. Redesign the beam in S355 steel for the new loading.

18.1.1 Loading (factored)

Factored compression force $F_c = 2400$ kN
Major axis point load $W = 100$ kN
Major axis distributed load $w = 2$ kN/m (assumed self-weight)
Minor axis distributed load $w = 3$ kN/m

18.1.2 Bending moment and shear force

Major axis

The shear force and bending moment diagrams for major axis bending are as shown in Figure 18.1.

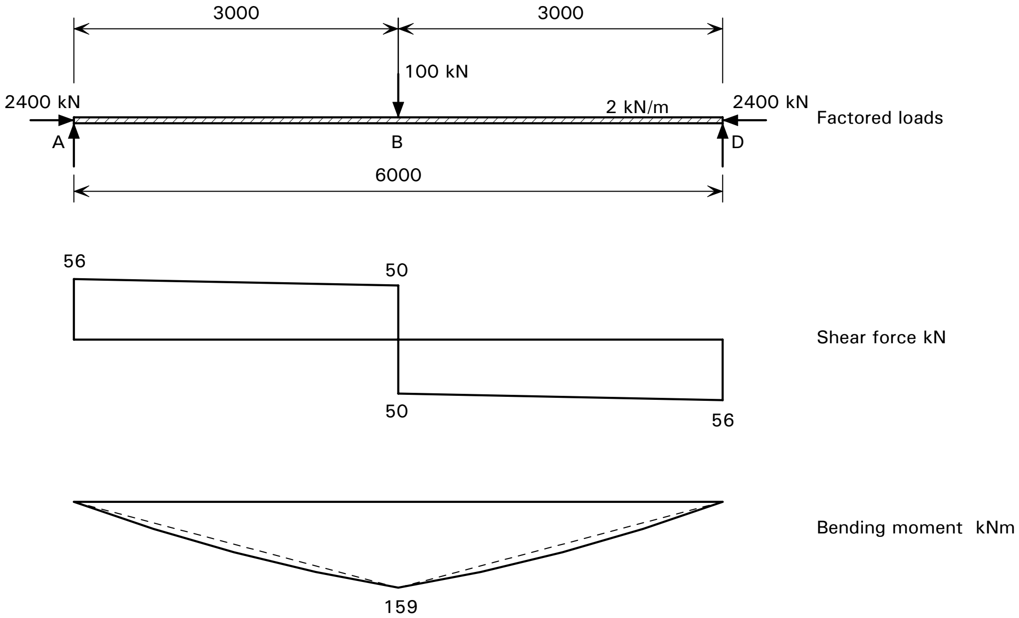

Figure 18.1 Major axis bending

Maximum moment occurs at the centre:

$$M = \frac{wL^2}{8} + \frac{WL}{4} = \frac{2.0 \times 6.0^2}{8} + \frac{100 \times 6.0}{4} = 159 \text{ kNm}$$

Example 18 Biaxial bending and compression (slender section)	Sheet 2 of 9	Rev

Shear force at the ends:

$$F_{\mathrm{ve}} = \frac{wL}{2} + \frac{W}{2} \quad = \frac{2.0 \times 6.0}{2} + \frac{100}{2} \quad = 56 \text{ kN}$$

Shear force at the centre:

$$F_{\mathrm{vc}} = F_{\mathrm{ve}} - \frac{wL}{2} \quad = 50 \text{ kN}$$

Minor axis

The shear force and bending moment diagrams for minor axis bending are as shown in Figure 18.2.

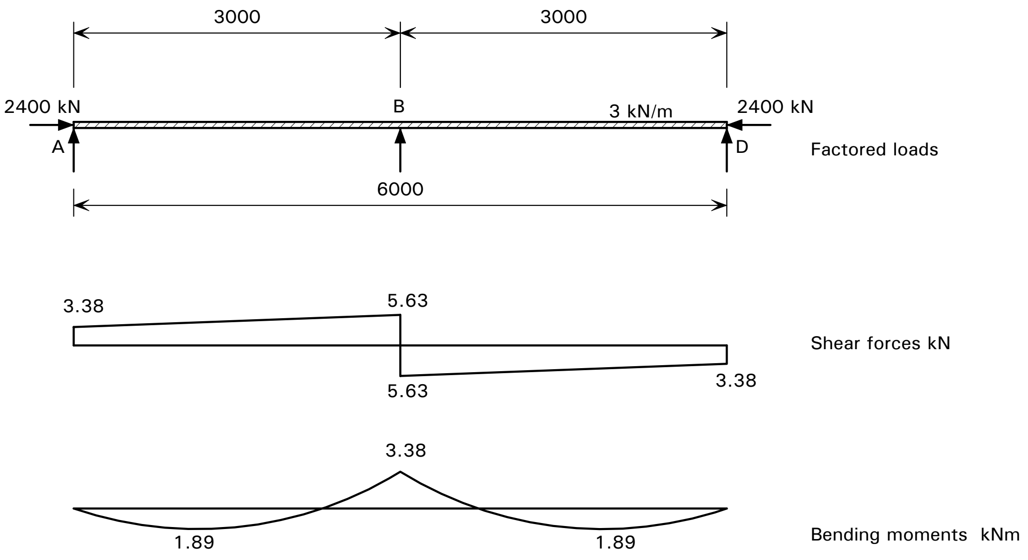

Figure 18.2 Minor axis bending

Maximum moment occurs at the central support:

$$M = \frac{wL^2}{8} \quad = \frac{3.0 \times 3.0^2}{8} \quad = 3.38 \text{ kNm}$$

From the shear force diagram, the maximum shear force also occurs at the central support.

$F_{\mathrm{v}} = 5.63$ kN

Example 18 Biaxial bending and compression (slender section)	Sheet 3 of 9	Rev

18.2 Member checks

18.2.1 Trial section

Try 610 x 229 x 101 UB in grade S355

From section property tables:

Depth	D	= 602.6 mm	Vol 1 Page B-2
Width	B	= 227.6 mm	
Web thickness	t	= 10.5 mm	
Flange thickness	T	= 14.8 mm	
Depth between fillets	d	= 547.6 mm	Vol 1 Page B-3
Area of cross-section	A_g	= 129 cm^2	
Plastic modulus	S_x	= 2880 cm^3	
Plastic modulus	S_y	= 400cm^3	
Elastic modulus	Z_x	= 2520 cm^3	
Elastic modulus	Z_y	= 256 cm^3	
Radius of gyration	r_x	= 24.2 cm	
Radius of gyration	r_y	= 4.75 cm	
Buckling parameter	u	= 0.863	
Torsional index	x	= 43.1	

Local buckling ratios: — Vol 1 Page B-3

Flange b/T = 7.69

Web d/t = 52.2

18.2.2 Section classification

Grade of steel = S355 — 3.1.1 Table 9

T < 16 mm

Therefore p_y = 355 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{355}} = 0.88$$

For the outstand element of a compression flange, the limiting b/T for a class 1 plastic flange is 9ε. Limiting b/T = 9×0.88 = 7.92 — 3.5.2 Table 11

The actual b/T = 7.69 < 7.92

Therefore, **the flange is class 1 plastic**.

For the web of an I-or H-section under axial compression and bending (the "generally" case in Table 11), the limiting d/t for a class 1 plastic web is — 3.5.2 Table 11

$$\frac{80\varepsilon}{1+r_1} \text{ but } \geq 40\varepsilon$$

3.5.5

$$r_1 = \frac{F_c}{dtp_y} = \frac{2400 \times 10^3}{547.6 \times 10.5 \times 355} = 1.18 \text{, but } -1 < r_1 \leq 1 \text{, therefore } r_1 = 1.0$$

$$\text{Limiting } d/t = \frac{80 \times 0.88}{1+1} = 35.2$$

The actual d/t = 52.2 > 35.2

Therefore, **the web is not class 1 plastic.**

The limiting d/t for a class 2 compact web is $\dfrac{100\varepsilon}{1+1.5r_1}$ but $\geq 40\varepsilon$ — 3.5.2 Table 11

$$\text{Limiting } d/t = \frac{100\times0.88}{1+(1.5\times1.0)} = 35.2$$

The actual d/t = 52.2 > 35.2
Therefore, the **web is not class 2 compact.**

The limiting d/t for a class 3 semi-compact web is $\dfrac{120\varepsilon}{1+2r_2}$ but $\geq 40\varepsilon$ — 3.5.2 Table 11

$$r_2 = \frac{F_c}{A_g p_y} = \frac{2400\times10^3}{12900\times355} = 0.52$$ 3.5.5

$$\text{Limiting } d/t = \frac{120\times0.88}{1+(2\times0.52)} = 51.8$$

The actual d/t = 52.2 > 51.8
Therefore, **the web is class 4 slender.**

Since the flange is class 1 plastic and the web is class 4 slender**, the cross section is class 4 slender**.

18.2.3 Determine the effective length

The member is pinned at both ends about both axes, with an additional restraint against lateral (minor axis) deflection at its mid-span.

L_{Ex} = 6000 mm

L_{Ey} = 3000 mm

18.2.4 Slenderness

4.7.2

$$\lambda_x = \frac{L_{Ex}}{r_x} = \frac{6000}{242} = 24.8$$

$$\lambda_y = \frac{L_{Ey}}{r_y} = \frac{3000}{47.5} = 63.2$$

18.2.5 Check the compression resistance

Basic requirement $F_c \leq P_c$ — 4.7.4 Vol 1 Page B-62

P_c = $A_{eff} p_c$

A_{eff} = 110 cm^2

The compressive strength p_c is obtained from the relevant strut curve for buckling about the x-x and y-y axes using the appropriate reduced slenderness in each case.

Buckling about the x-x axis

Use strut curve a — Table 23

$$\text{Reduced slenderness} = \lambda_x\sqrt{\frac{A_{eff}}{A_g}} = 24.8\times\sqrt{\frac{110}{129}} = 22.9$$ 4.7.4

Example 18 Biaxial bending and compression (slender section)	Sheet 5 of 9	Rev

For λ = 22.9 and p_y = 355 N/mm², p_{cx} = 349 N/mm² — Table 24a

P_{cx} = 11000 × 349 × 10^{-3} = 3839 kN

Buckling about the y-y axis

Use strut curve b — Table 23

Reduced slenderness $= \lambda_y \sqrt{\dfrac{A_{eff}}{A_g}} = 63.2 \times \sqrt{\dfrac{110}{129}} = 58.4$ — 4.7.4

For λ = 58.4 and p_y = 355 N/mm², p_{cy} = 277 N/mm² — Table 24b

P_{cy} = 11000 × 277 × 10^{-3} = 3047 kN

P_c = min(P_{cx}, P_{cy}) = 3047 kN

F_c = 2400 kN

2400 kN < 3047 kN

Therefore, **the compression resistance is adequate.**

18.2.6 Check for shear buckling

If the d/t ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000[1]. — 4.2.3

70ε = 70 × 0.88 = 61.6

d/t = 52.2 < 61.6, so **there is no need to check for shear buckling**.

18.2.7 Check the shear capacity

Major axis

Basic requirement $F_v \le P_v$ — 4.2.3

P_v = 0.6 $p_y A_v$

p_y = 355 N/mm²

A_v = tD = 10.5 × 602.6 = 6327 mm²

P_v = 0.6 × 355 × 6327 × 10^{-3} = 1348 kN

Maximum shear force F_v = 56 kN

56 kN < 1348 kN

Therefore, **the major axis shear capacity is adequate.**

Minor axis

Basic requirement $F_v \le P_v$ — 4.2.3

P_v = 0.6 $p_y A_v$

p_y = 355 N/mm²

A_v = 0.9 A_o, where A_o is the combined area of the two flanges.

A_v = 0.9 × 2 × 227.6 × 14.8 = 6063 mm²

P_v = 0.6 × 355 × 6063 × 10^{-3} = 1291 kN

Maximum shear force F_v = 5.63 kN.

5.63 kN < 1291 kN

Therefore, **the minor axis shear capacity is adequate.**

18.2.8 Check the moment capacity

Major axis

Basic requirement $M_x \le M_{cx}$ — 4.2.5.2

By inspection the shear is low (i.e. F_v < 0.6 P_v)

For low shear, the moment capacity for a class 4 section is given by: — 4.2.5.2

M_{cx} = $p_y Z_{x,eff}$

However, when the cross section is non-slender under pure bending (i.e. it is only class 4 in the presence of a high axial load), $Z_{x,eff}$ is taken as equal to Z_x. 3.6.2.3

M_{cx} = 355 × 2520 × 10^{-3} = 895 kNm
From Figure 18.1, M_x = 159 kNm
159 kNm < 895 kNm
Therefore, **the major axis moment capacity is adequate.**

Minor axis

Basic requirement $M_y \le M_{cy}$ 4.2.5.2
By inspection the shear is low (i.e. $F_v < 0.6\ P_v$).
For low shear, the moment capacity for a class 4 section is given by:

$M_{cy} = p_y Z_{y,eff}$ 4.2.5.2

Once again, Z_{eff} is taken as equal to Z, since the section is only class 4 due to the axial load. 3.6.2.3

M_{cy} = 355 × 256 × 10^{-3} = 90.9 kNm
From Figure18.2, M_y = 3.38 kNm
3.38 kNm < 90.9 kNm
Therefore, **the minor axis moment capacity is adequate.**

18.2.9 Lateral-torsional buckling

The beam is restrained against lateral-torsional buckling at its supports and at its mid-span. There are, therefore, two segments to consider. However, due to symmetry, it is sufficient to design one segment only in this example. Consider segment length AB.

Basic requirements: $M_x \le M_b / m_{LT}$ 4.3.6.2
and $M_x \le M_{cx}$ (already checked in 18.2.8 above)

The buckling resistance moment M_b for a class 4 slender section is given by:

$M_b = p_b Z_{x,eff}$ 4.3.6.4

$Z_{x,eff}$ is taken as equal to Z_x, since the cross section would be non-slender under pure bending.

p_b is the bending strength and is dependent on the design strength p_y and the equivalent slenderness λ_{LT}.

$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$ 4.3.6.7

$\lambda/x = \dfrac{63.2}{43.1} = 1.5$

For a section with equal flanges and λ/x = 1.5, v = 0.97 Table 19

$\beta_w = \dfrac{Z_x}{S_x} = \dfrac{2520}{2880} = 0.875$ $\sqrt{\beta_w} = 0.935$ 4.3.6.9

Therefore, λ_{LT} = 0.863 × 0.97 × 63.2 × 0.935 = 49.5
For p_y = 355 N/mm² and λ_{LT} = 49.5, Table 16 gives p_b = 294 N/mm² Table 16
M_b = 294 × 2520 × 10^{-3} = 741 kNm

Since there is no major axis loading between the restraints (apart from the self-weight of the beam, which is considered insignificant for the purpose of evaluating m_{LT}), m_{LT} can be obtained directly from Table 18 for a known value of β.

β is the ratio of the bending moments at points A and B, i.e. $\beta = 0$ 4.3.6.6 Table 18

From Table 18, $m_{LT} = 0.6$

$M_b / m_{LT} = 741 / 0.6 = 1235$ kNm

From Figure 18.2, $M_x = 159$ kNm

159 kNm < 1235 kNm

Therefore, **the buckling resistance moment is adequate.**

18.2.10 Interaction between axial load and bending

Cross section capacity

Basic requirement: $\dfrac{F_c}{A_{eff}\, p_y} + \dfrac{M_x}{M_{cx}} + \dfrac{M_y}{M_{cy}} \leq 1$ 4.8.3.2

At point B

$$\frac{2400 \times 10^3}{110 \times 10^2 \times 355} + \frac{159}{895} + \frac{3.38}{90.9}$$

$= 0.615 + 0.178 + 0.037 = 0.830 < 1$

Therefore, **the cross section capacity is adequate.**

Member buckling resistance

BS 5950-1:2000 presents two methods for checking the member buckling resistance: the simplified method (clause 4.8.3.3.1) and the more exact method (clause 4.8.3.3.2). In the simplified method, the following relationships must be satisfied:

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \leq 1 \quad \text{and}$$ 4.8.3.3.1

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \leq 1$$ 4.8.3.3.1

M_{LT} is the maximum major axis moment in segment AB = 159 kNm. 4.8.3.3.4

$m_x = 0.9$ and $m_y = 0.55$ (see Example 16) Table 26

$$\frac{2400}{3047} + \frac{0.9 \times 159 \times 10^6}{355 \times 2520 \times 10^3} + \frac{0.55 \times 3.38 \times 10^6}{355 \times 256 \times 10^3}$$

$= 0.788 + 0.160 + 0.020 = 0.97 < 1$

$$\frac{2400}{3047} + \frac{0.6 \times 159}{741} + \frac{0.55 \times 3.38 \times 10^6}{355 \times 256 \times 10^3}$$

$= 0.788 + 0.129 + 0.020 = 0.94 < 1$

Therefore, **the member buckling resistance is adequate.**

<u>Adopt 610 x 229 x 101 UB in S355</u>

Note: 1. If appropriate, the web should be checked for bearing and buckling at the supports and at the point of load application, as in Example 2.

2. The deflections should be checked at the serviceability limit state in accordance with the recommendations in clause 2.5.2.

Example 18 Biaxial bending and compression (slender section)	Sheet 8 of 9	Rev

18.3 Blue Book approach

The section capacities and member resistances calculated in Section 18.2 above could have been obtained directly from Volume 1[2].

18.3.1 Compression resistance

Vol 1 Page D-3

For L_{Ex} = 6.0 m, P_{cx} = 3850 kN
For L_{Ey} = 3.0 m, P_{cy} = 3050 kN
Therefore, P_c = 3050 kN
F_c = 2400 kN
2400 kN < 3050 kN
Therefore, **the compression resistance is adequate.**

18.3.2 Shear Capacity (major axis)

Vol 1 Page D-78

P_v = 1350 kN (no value given for minor axis shear)
F_v = 56 kN
56 kN < 1350 kN
Therefore, **the major axis shear capacity is adequate.**

18.3.3 Moment capacity

Vol 1 Page D-108

For F/P_z 0.52
M_{cx} = 407 kNm
M_x = 159 kNm
159 kNm < 407 kNm
Therefore, **the major axis moment capacity is adequate.**

Vol 1 Page D-108

For F/P_z 0.52
M_{cy} = 41.4 kNm
M_y = 3.38 kNm
3.38 kNm < 41.4 kNm
Therefore, **the minor axis moment capacity is adequate.**

18.3.4 Buckling resistance moment

Volume 1 (Page D-109) does not include M_b values for class 4 slender sections.
From 18.2.9 M_b = 741 kNm

18.3.5 Interaction between axial load and bending

Cross section capacity

$$\frac{F_c}{A_{eff}\, p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} = \frac{2400 \times 10^3}{110 \times 10^2 \times 355} + \frac{159}{407} + \frac{3.38}{41.4}$$

= 0.615 + 0.391 + 0.082 = 1.09 > 1
Therefore, **the cross section capacity is NOT adequate.**

For slender sections, the moment capacities in Volume 1 are conservative because they have been based on reduced design strength and gross section properties (i.e. $p_{yr} Z$) instead of $p_y Z_{eff}$.

However, the full calculation in Section 18.2.10 shows that the *cross section is actually adequate in this case.*

Example 18 Biaxial bending and compression (slender section)	Sheet 9 of 9	Rev

Member buckling resistance — 4.8.3.3.4

For a quick approximation, m_x, m_y and m_{LT} may conservatively be taken as 1.0. However, more accurately, m_x and m_y should be obtained from Table 26 of BS 5950-1:2000 and m_{LT} should be obtained from Table 18 of BS 5950-1:2000.

m_x = 0.9 — Table 26

m_y = 0.55

m_{LT} = 0.6 — Table 18

$p_y Z_x$ = 893 kNm — Vol 1

$p_y Z_y$ = 90.9 kNm — Page D-109

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} = \frac{2400}{3050} + \frac{0.9 \times 159}{893} + \frac{0.55 \times 3.38}{90.9}$$

$$= 0.787 + 0.160 + 0.020 \quad = 0.97 < 1$$

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} = \frac{2400}{3050} + \frac{0.6 \times 159}{741} + \frac{0.55 \times 3.38}{90.9}$$

$$= 0.787 + 0.129 + 0.020 \quad = 0.94 < 1$$

Therefore, **the member buckling resistance is adequate.**

Adopt 610 x 229 x 101 UB in S355

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CALCULATION SHEET

Job No. CDS 153		Sheet 1 of 5	Rev
Job Title Example no. 19			
Subject Lattice beam			
Client SCI	Made by MDH	Date Jun 2003	
	Checked by ASM	Date Oct 2003	

19 Lattice beam

19.1 Introduction

The top chord of a lattice girder supports purlins at A, B and C, as shown in Figure 19.1, resulting in a factored point load of 11.2 kN at each location. The member is a 150 × 150 × 5 hot finished RHS in S275 steel and is continuous at A, B and C. Check the adequacy of the top chord by considering its section capacity and buckling resistance.

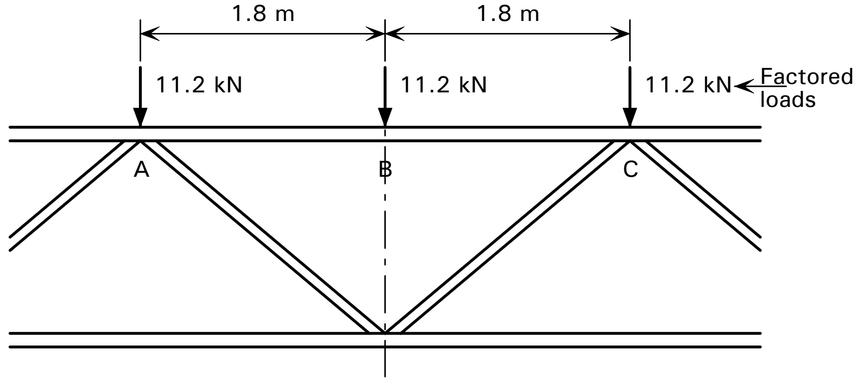

Figure 19.1

19.1.1 Loading

The factored compressive force in ABC = 598 kN.
The factored bending moment diagram is shown in Figure 19.2 (moments in kNm).

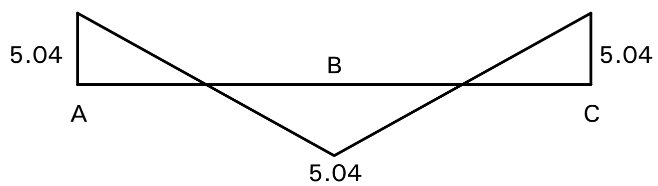

Figure 19.2

19.1.2 Section properties

Depth	D	= 150 mm
Thickness	t	= 5.0 mm
Area of section	A	= 28.7 cm^2
Radius of gyration	r	= 5.90 cm
Plastic modulus	S	= 156 cm^3
Elastic modulus	Z	= 134 cm^3
Local buckling ratio	d/t	= 27.0

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Page B-22

Example 19 Lattice beam	Sheet 2 of 6	Rev

19.2 Member design

19.2.1 Classify the cross section

Grade of steel = S275 — 3.1.1
t < 16.0 mm — Table 9
Therefore p_y = 275 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{275}} = 1.0$$

For the flange of a HF RHS, the limiting b/t is 28ε, but $\leq 80\varepsilon - d/t$ — 3.5.2
Limiting b/t = 28ε = 28×1.0 = 28.0 — Table 12
The actual b/t = d/t = 27.0 < 28.0
Therefore, **the flange is class 1 plastic**.

For the web of a HF RHS, the limiting d/t is at least 40ε. Therefore, for a square hollow section, where b/t is always equal to d/t, the classification of the cross section will always be determined by the classification of the flange and there is no need to check the web. — 3.5.2 Table 12
Therefore, **the cross section is class 1 plastic**.

19.2.2 Determine the effective lengths

In the plane of the girder, there is no vertical restraint at B and the effective length can be assumed to be — 4.7.3 Table 22
L_{Ex} = $0.85L_{AC}$ = 0.85×3.6 = 3.06 m

Out of plane, the purlin at B provides restraint to the top chord and the effective length can be assumed to be
L_{Ey} = $1.0L_{AB}$ = 1.0×1.8 = 1.8 m

19.2.3 Slenderness

4.7.2

$$\lambda_x = \frac{L_{Ex}}{r} = \frac{3060}{59} = 51.9$$

$$\lambda_y = \frac{L_{Ey}}{r} = \frac{1800}{59} = 30.5$$

19.2.4 Check the compression resistance

Basic requirement $F_c \leq P_c$ — 4.7.4
$P_c = A_g p_c$ (for a class 1 plastic cross section)

The compressive strength p_c is obtained from the relevant strut curve for buckling about the x-x and y-y axes.

Buckling about the x-x axis

Use strut curve a — Table 23
For λ_x = 51.9 and p_y = 275 N/mm^2, p_{cx} = 249 N/mm^2 — Table 24a
P_{cx} = $2870 \times 249 \times 10^{-3}$ = 715 kN

Example 19 Lattice beam	Sheet 3 of 6	Rev

Buckling about the y-y axis

Use strut curve a — Table 23

For $\lambda_y = 30.5$ and $p_y = 275$ N/mm^2, $p_{cy} = 266$ N/mm^2 — Table 24a

$P_{cy} = 2870 \times 266 \times 10^{-3} = 763$ kN

P_c is the lesser of P_{cx} and $P_{cy} = 715$ kN

$F_c = 598$ kN

598 kN < 715 kN

Therefore, **the compression resistance is adequate.**

19.2.5 Check for shear buckling

If the d/t ratio exceeds 70ε for a rolled section, the web should be checked for shear buckling in accordance with clause 4.4.5 of BS 5950-1:2000[1]. — 4.2.3

$70\varepsilon = 70 \times 1.0 = 70.0$

In this case, $d/t = 27.0 < 70\varepsilon$, so **there is no need to check for shear buckling**.

19.2.6 Check the shear capacity

Basic requirement $F_v \leq P_v$ — 4.2.3

$P_v = 0.6\, p_y A_v$

$p_y = 275$ N/mm^2

$$A_v = \left(\frac{AD}{D+B}\right) = \frac{2870 \times 150}{150 + 150} = 1435 \text{ mm}^2$$

$P_v = 0.6 \times 275 \times 1435 \times 10^{-3} = 237$ kN

$F_v = 11.2$ kN

11.2 kN < 237 kN

Therefore, **the shear capacity is adequate.**

19.2.7 Check the moment capacity

Basic requirement $M_x \leq M_{cx}$ — 4.2.5.2

Without calculation, it is clear that the shear is "low" in this case (i.e. $F_v < 0.6\, P_v$).

For low shear, the moment capacity for a class 1 section is given by: — 4.2.5.2

$M_{cx} = p_y S_x$

$M_{cx} = 275 \times 156 \times 10^{-3} = 42.9$ kNm

Check limit to avoid irreversible deformation under serviceability loads. — 4.2.5.1

For a continuous beam $M_{cx} \leq 1.5\, p_y Z_x$

$1.5 p_{yx} = 1.5 \times 275 \times 134 \times 10{-}3 = 55.3$ kNm

Therefore $M_{cx} = 42.9$ kNm

From Figure 19.2, $M_x = 5.04$ kNm.

5.04 kNm < 42.9 kNm

Therefore, **the moment capacity is adequate.**

19.2.8 Lateral-torsional buckling

For square hollow sections, there is no need to consider lateral-torsional buckling and M_b is given by: — 4.3.6.1

$M_b = p_y S_x = 42.9$ kNm

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19.2.9 Interaction between axial load and bending

Cross section capacity

Basic requirement: $\dfrac{F_c}{A_g p_y} + \dfrac{M_x}{M_{cx}} + \dfrac{M_y}{M_{cy}} \le 1$ 4.8.3.2

At point B: $\dfrac{598 \times 10^3}{2870 \times 275} + \dfrac{5.04}{42.9} + 0 \quad = 0.758 + 0.117 + 0 \quad = 0.88 < 1$

Therefore, **the cross section capacity is adequate.**

Member buckling resistance

In the simplified method, the following relationships must be satisfied:

$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \le 1 \text{ and}$$ 4.8.3.3.1

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \le 1$$

P_c is the smaller of P_{cx} and P_{cy}

M_x is the maximum major axis moment in segment AC = 5.04 kNm

M_{LT} is the maximum major axis moment in segment AB = 5.04 kNm

m_x is determined between restraints on the x axis, i.e. A and C, according to the shape of the bending moment diagram (see Figure 19.2).

$$m_x = 0.2 + \frac{0.1 M_2 + 0.6 M_3 + 0.1 M_4}{M_{max}} \quad \text{but } m_x \ge \frac{0.8 M_{24}}{M_{max}}$$ 4.8.3.3.4 Table 26

The moments M_2 and M_4 are the values at the quarter points and the moment M_3 is the value at mid-length. M_{max} is the maximum moment in the segment and M_{24} is the maximum moment in the central half of the segment.

Location	M_2	M_3	M_4	M_{24}	M_{max}
Moment kNm	0	5.04	0	5.04	5.04

$$m_x = 0.2 + \frac{0 + 0.6 \times 5.04 + 0}{5.04} = 0.8 \quad \text{but } m_x \ge \frac{0.8 \times 5.04}{5.04} = 0.8$$

Therefore, $m_x = 0.8$ 4.3.6.6 Table 18

m_{LT} is determined between restraints on the y-y axis, i.e. A and B (or B and C)

$M = 5.04$, $\beta M = -5.04$, therefore $\beta = -1$ and $m_{LT} = 0.44$

$$\frac{598}{715} + \frac{0.8 \times 5.04 \times 10^6}{275 \times 134 \times 10^3} + 0 \quad = 0.836 + 0.109 + 0 \quad = 0.95 < 1$$ 4.8.3.3.1

$$\frac{598}{763} + \frac{0.44 \times 5.04}{42.9} + 0 = 0.784 + 0.052 + 0 \quad = 0.84 < 1$$ 4.8.3.3.1

Therefore, **the member buckling resistance is adequate.**

Note: The deflections should be checked as in Example 2.

Adopt 150 x 150 x 5 HF RHS in S275 steel

19.3 Blue Book approach

The member capacities and resistances calculated in section 19.2 could have been obtained directly from Volume 1[2].

19.3.1 Compression resistance

For L_{Ex} = 3.06 m, P_{cx} = 714 kN
For L_{Ey} = 1.8 m, P_{cy} = 759 kN (for 2m)
Therefore, P_c = 714 kN
F_c = 598 kN
598 kN < 714 kN
Therefore, **the compression resistance is adequate.**

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19.3.2 Shear capacity

P_v = 237 kN
F_v = 11.2 kN
11.2 kN < 237 kN
Therefore, **the shear capacity is adequate.**

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19.3.3 Moment capacity

M_{cx} = 42.9 kNm
M_x = 5.04 kNm
5.04 kNm < 42.9 kNm
Therefore, **the moment capacity is adequate.**

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19.3.4 Interaction between axial load and bending

Cross section capacity

$A_g p_y$ = 789 kN

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$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} = \frac{598}{789} + \frac{5.04}{42.9} + 0 = 0.758 + 0.117 = 0.88 < 1$$

Therefore, **the cross section capacity is adequate.**

Member buckling resistance

For a quick approximation, m_x and m_{LT} may conservatively be taken as 1.0. However, more accurately, m_x should be obtained from Table 26 of BS 5950-1:2000 and m_{LT} should be obtained from Table 18 of BS 5950-1:2000.

m_x = 0.8
m_{LT} = 0.44

Table 26
Table 18

$p_y Z$ = 36.9 kNm

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$$\frac{F_c}{P_c} + \frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} = \frac{598}{714} + \frac{0.8 \times 5.04}{36.9} + 0$$

$$= 0.838 + 0.109 + 0 = 0.95 < 1$$

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For a square hollow section, $M_b = M_c = 42.9$ kN

$$\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} = \frac{598}{759} + \frac{0.44 \times 5.04}{42.9} + 0$$

$= 0.788 + 0.052 + 0 \quad = 0.84 < 1$

Therefore, **the member buckling resistance is adequate.**

Adopt 150 x 150 x 5 HF RHS in S275 steel

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CALCULATION SHEET

Job No. CDS 153		Sheet 1 of 22	Rev
Job Title Example no. 20			
Subject Gantry girder, using a UB with a plated top flange			
Client SCI	Made by JBL	Date Jun 2003	
	Checked by ASM	Date Oct 2003	

20 Gantry girder, using a UB with a plated top flange

20.1 Introduction

Design a simply supported gantry girder, of span 8.0 m, using a UB and a top flange plate. The gantry girder is to be used in a building to carry an indoor overhead travelling crane for a medium to heavy workshop. The crane is class Q3 as defined in BS 2573-1:1983[4]. The wheels of the crane are flanged on both sides.

The crane details (Figure 20.1) are as follows:

Crane capacity (hook load)	W_{cap}	= 100 kN
Weight of crab	W_{cb}	= 20 kN
Weight of crane bridge (including end carriages)	W_c	= 80 kN
Weight of gantry girder	W_G	= 15 kN
Span of crane bridge	L_c	= 15 m
Wheel spacing in end carriage	a_w	= 4 m
Minimum hook approach	a_h	= 1 m

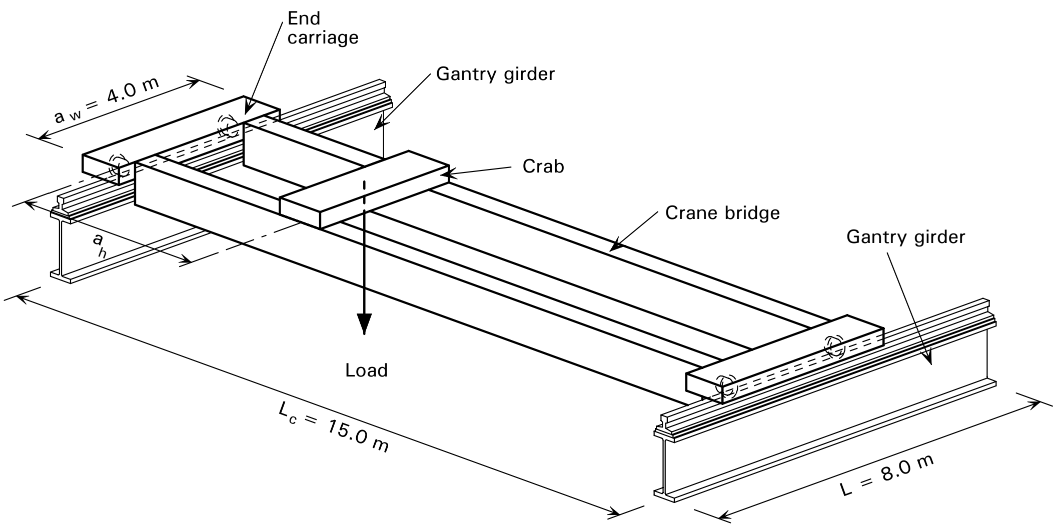


Figure 20.1 *Arrangement of gantry girder and crane*

20.1.1 Wheel loads

Note: Section 20.6 of this example gives background and information to gantry girder loading

Vertical wheel load

Maximum unfactored static load (per wheel), W_{us} (from the crane bridge self-weight plus crab self-weight plus hook load).

$$W_{us} = \frac{1}{2}\left(\frac{W_c}{2} + (W_{cb} + W_{cap})\frac{L_c - a_h}{L_c}\right) = \frac{1}{2}\times\left(40 + (20+100)\times\frac{14}{15}\right) = 76 \text{ kN}$$

Impact factor 1.3

BS 2573-1[4] Table 4

Maximum unfactored dynamic vertical load (per wheel) (see Table 20.5 of this example).

$$W_w = 1.3W_{us} = 1.3\times 76 = 98.8 \text{ kN}$$

Horizontal wheel load

Horizontal loads that act both laterally (i.e. transverse) and longitudinally to the rail of the girder need to be considered. Such horizontal loads are the result of surge and crabbing.

Horizontal forces due to surge

This is the same as inertia forces (clause 3.1.5.1 of BS 2573-1:1983[4]).
Transverse surge forces (see Figure 20.2) are developed due to:

1) the acceleration and braking of the crab when moving along the crane bridge

2) lateral pulling by the crab when lifting a load, or swinging of the load.

Longitudinal surge forces (acting along the rails) result from acceleration and braking of the crane bridge and to longitudinal pulling on the lifted loads.

The value of the transverse surge load is taken as 10% of the sum of the crab weight and the lifted load. Where the wheels have single flanges, the transverse surge is carried by only two wheels. Where the wheels have double flanges (i.e. one flange on each side of the wheel), the transverse surge is shared by all four wheels.
N.B. assuming double flanged wheels.

Unfactored horizontal transverse surge load per wheel, W_{H1}

$$W_{H1} = \frac{0.1(W_{cb} + W_{cap})}{No.\ of\ wheels} = \frac{0.1\times(20+100)}{4} = 3 \text{ kN}$$

Unfactored horizontal longitudinal surge load (along the rails) per wheel, W_{H2} (5% of the static load)

$$W_{H2} = 0.05W_{us} = 0.05\times 76 = 3.8 \text{ kN}$$

Horizontal forces due to crabbing

This is the same as skew loads due to travelling (caluse 3.1.5.2, BS 2573: Part 1:1983 [4])
Transverse crabbing forces (see Figure 20.3) are developed due to the oblique or skew travel of the crane bridge along the rail.

Unfactored crabbing force, transverse to the rail per wheel, F_R

4.11.2 (For class Q3 cranes)

$$F_R = \frac{L_c W_w}{40 a_w} \quad \text{but } F_R \geq \frac{W_w}{20}$$

$$F_R = \frac{15\times 98.8}{40\times 4} = 9.3 \text{ kN} \quad \text{but } \geq \frac{98.8}{20} = 4.9 \text{ kN}$$

Therefore, $F_R = 9.3$ kN

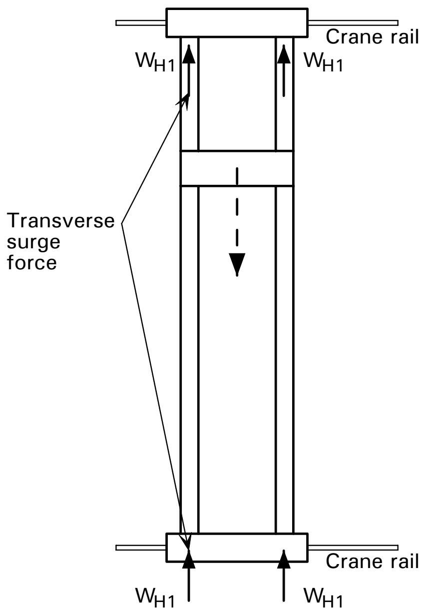


Figure 20.2 *Transverse surge force*

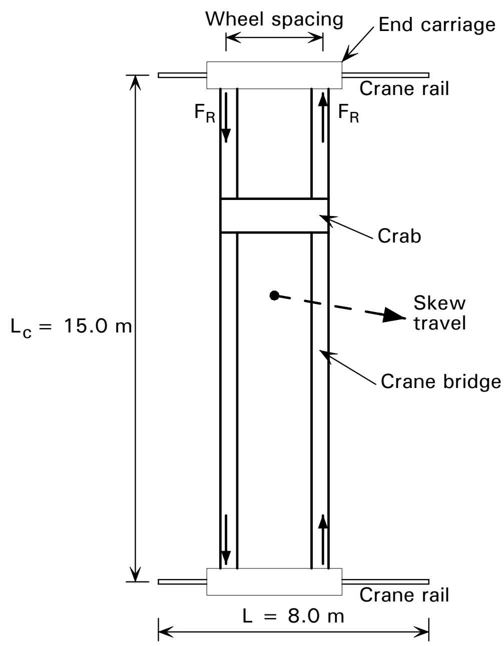


Figure 20.3 *Crabbing forces or skew loads due to travelling*

20.2 Design loads: bending moment and shear

The following three crane load combinations will be considered:

1. 1.4 Dead load + 1.6 Vertical crane loads
2. 1.4 Dead load + 1.6 Horizontal crane loads
3. 1.4 Dead load + 1.4 Vertical crane loads + 1.4 Horizontal crane loads

The maximum bending moments and shear forces for each load combination will be determined.

20.2.1 Crane load combination 1: 1.4 Dead load + 1.6 Vertical crane loads

Factored loads

Gantry girder dead load $W'_G = 1.4 \times W_G = 1.4 \times 15$ = 21 kN

Vertical crane load (per wheel) $W'_w = 1.6 \times W_w = 1.6 \times 98.8$ = 158 kN

The maximum bending moment and shear force will each need to be determined at two different wheel positions.

Maximum bending moment

Figure 20.4 shows the load, bending moment and shear force diagram corresponding to crane load combination 1. The position of the wheel loads result in the maximum bending moment.

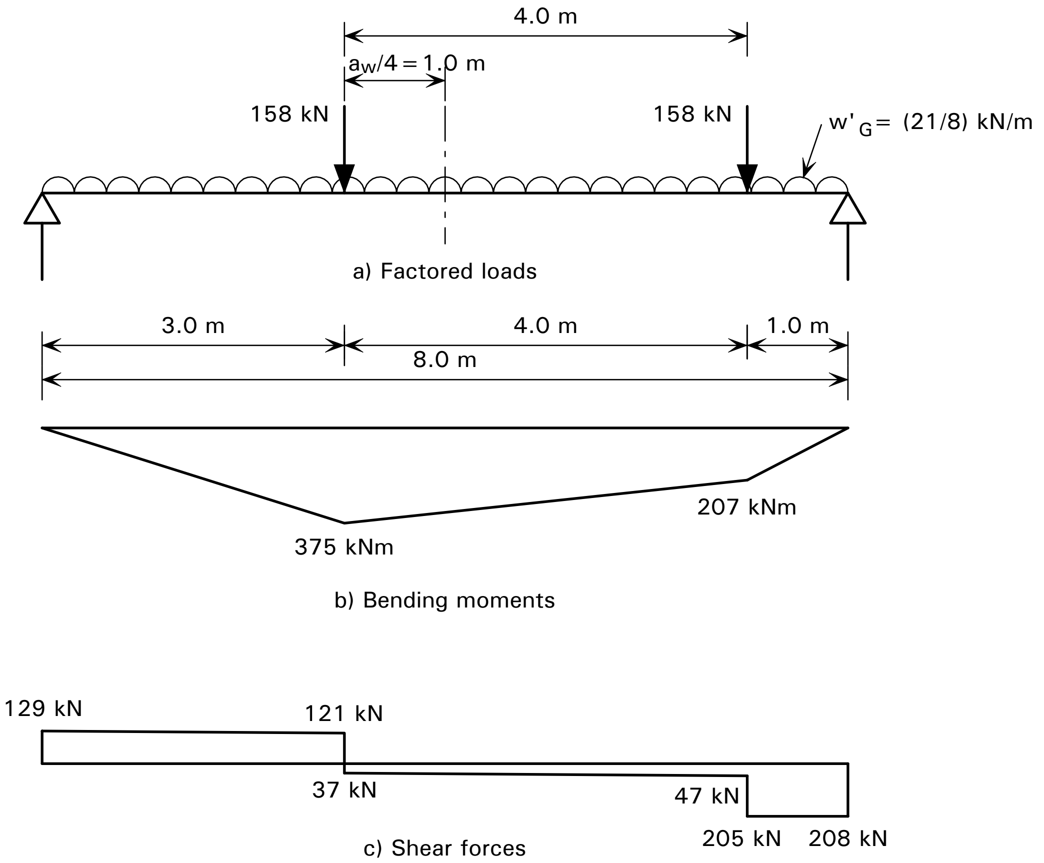


Figure 20.4 *Crane load combination 1, wheel position 1, for maximum bending moments*

Example 20 Gantry girder, using a UB with a plated top flange	Sheet 5 of 22	Rev

Maximum shear force

The maximum shear force occurs when one wheel is almost over a support as shown in Figure 20.5.

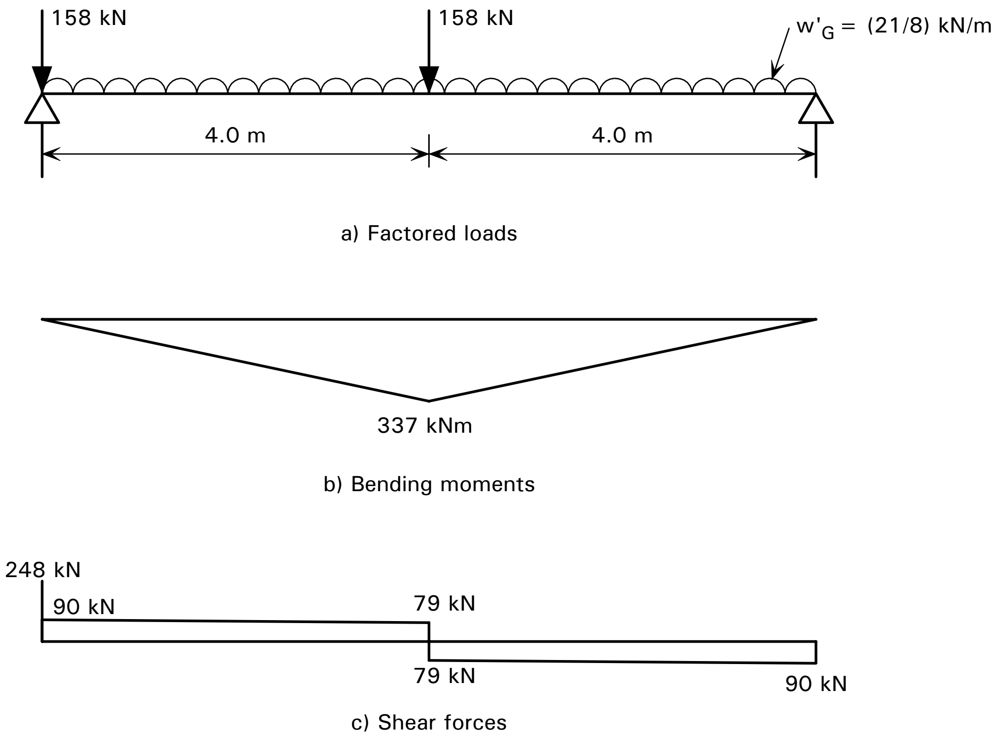


Figure 20.5 *Crane load combination 1, wheel position 2, for maximum shear force*

20.2.2 Crane load combination 2: 1.4 Dead load + 1.6 Horizontal crane loads

The crane load combinations in clause 2.4.1.3 of BS 5950-1:2000[1] are for "gantry girders and their supports". Crane load combination 2 is an important combination for the supports (e.g. longitudinal bracing resisting braking forces when the crane is in another bay) but is not a combination (i.e. horizontal crane loads without vertical crane loads) that can occur on an individual simply supported crane gantry girder. Therefore crane load combination 2 is not checked in this example.

20.2.3 Crane load combination 3: 1.4 Dead load + 1.4 Vertical crane loads + 1.4 Horizontal crane loads

To simplify the calculations and remain conservative the maximum bending moment from the vertical crane loads are combined with the maximum bending moments from the horizontal crane loads, even though these do not occur at the same position.

Factored loads

Gantry girder dead load $W'_G = 1.4 \times W'_G = 1.4 \times 15 = 21$ kN

Vertical crane load (per wheel) $W'_w = 1.4 \times W_w = 1.4 \times 98.8 = 138$ kN

Horizontal transverse force (per wheel) due to surge

$$W'_{H1} = 1.4 \times W_{H1} = 1.4 \times 3.0 = 4.2 \text{ kN}$$

Horizontal force (per wheel) due to crabbing.

$F'_R = 1.4 \times F_R = 1.4 \times 9.3 = 13.0$ kN

Bending moment for factored vertical loads

The maximum bending moment for vertical loads is as shown in Figure 20.6.

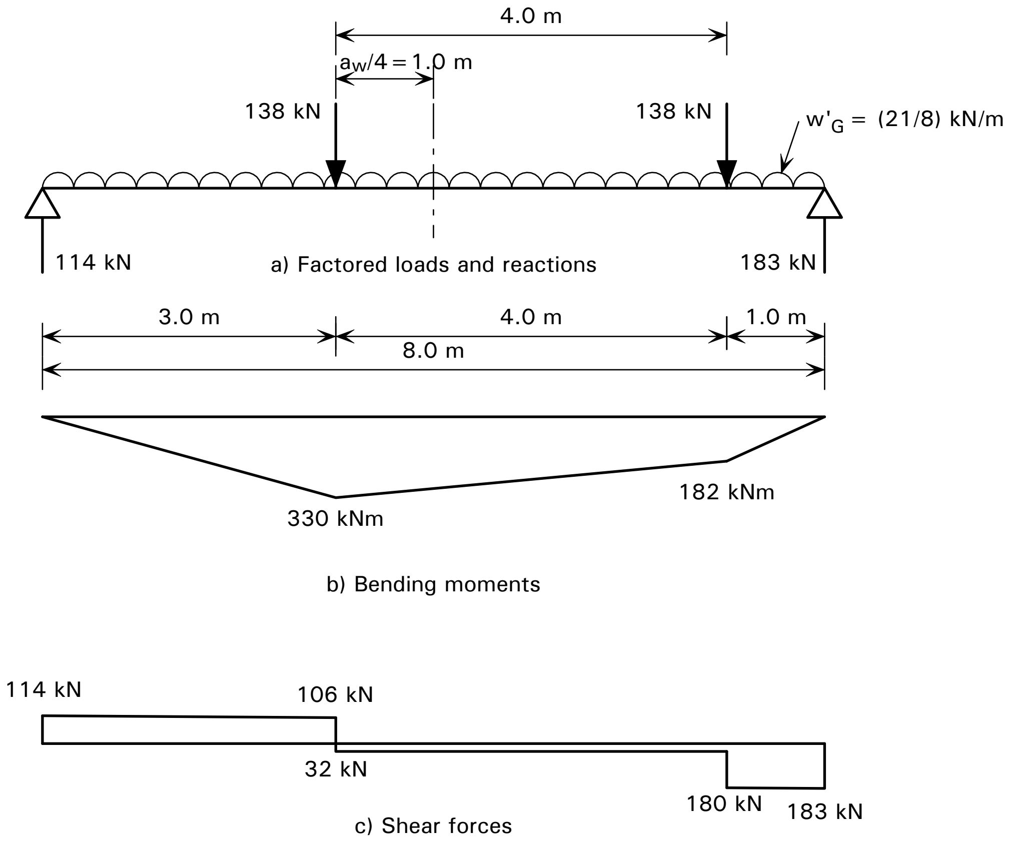


Figure 20.6 *Vertical loads (in crane load combination 3), for maximum bending moment*

Bending moment for factored horizontal loads

Worst case for bending moment for horizontal loads is as shown in Figure 20.7 for surge forces and Figure 20.8 for crabbing forces.

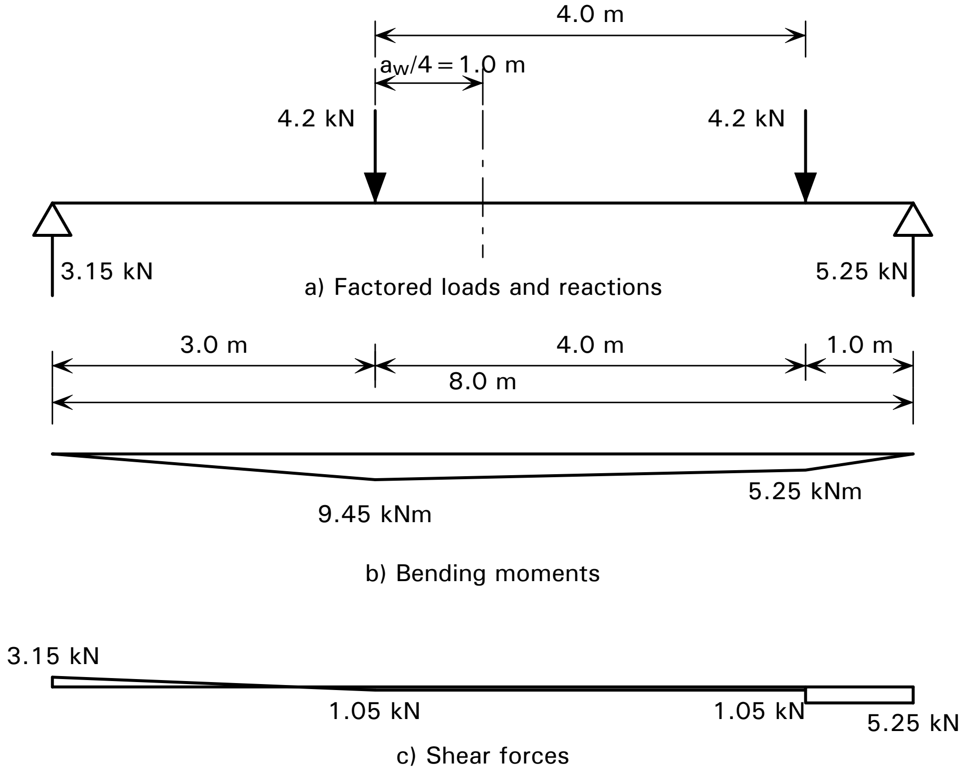


Figure 20.7 *Horizontal surge forces (in crane load combination 3), for maximum bending moment.*

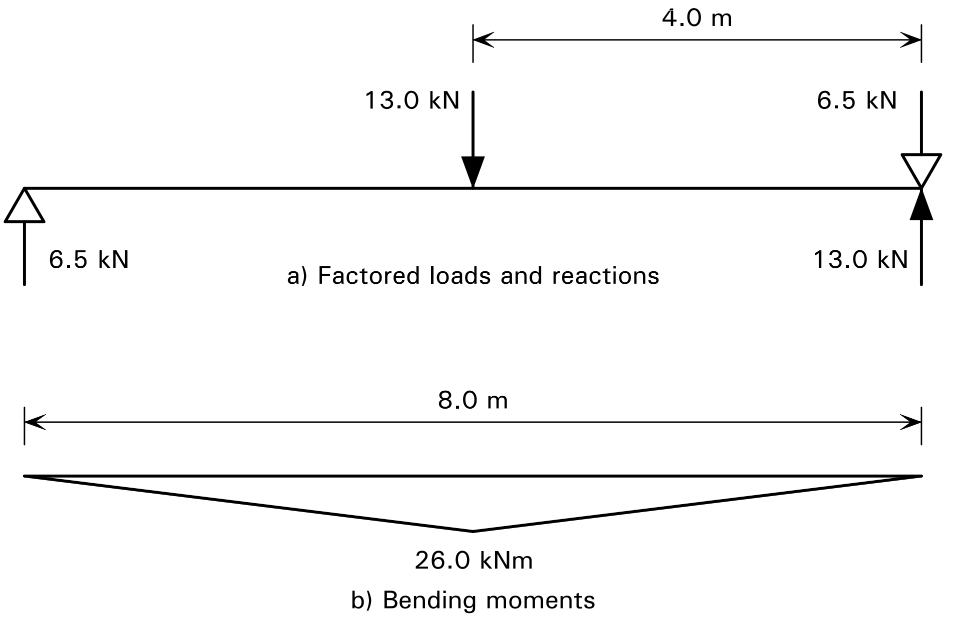


Figure 20.8 *Horizontal crabbing forces (in crane load combination 3), for maximum bending moment.*

Example 20 Gantry girder, using a UB with a plated top flange	Sheet 8 of 22	Rev

For crane load combination 3, the maximum horizontal bending moment due to crabbing (26.0 kNm from Figure 20.8) was larger than that due to surge (9.45 kNm from Figure 20.7). Therefore, for crane load combination 3, only the horizontal bending moment due to crabbing will be considered in sections 20.4.3 and 20.4.4.

20.3 Initial sizing of the gantry girder

This is normally a trial and error process, the UB section being chosen having a buckling resistance moment M_b approximating to the maximum vertical moment for $m_{LT} = 1$.

For this example try a 610 × 229 × 125 UB grade S275 with a 300 mm × 15 mm plate grade S275 steel. A top flange plate rather than a channel has been adopted, since welding is easier (see Figure20.9)

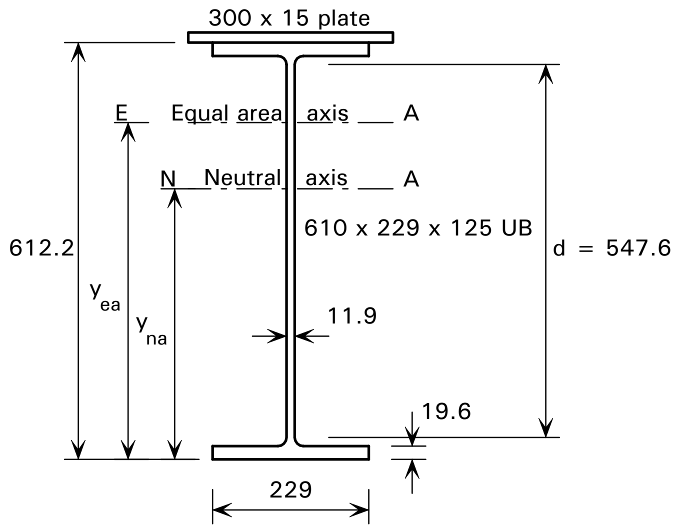


Figure 20.9 *Trial gantry girder section-UB with plate*

20.3.1 Section classification of compound section (UB plus plate)

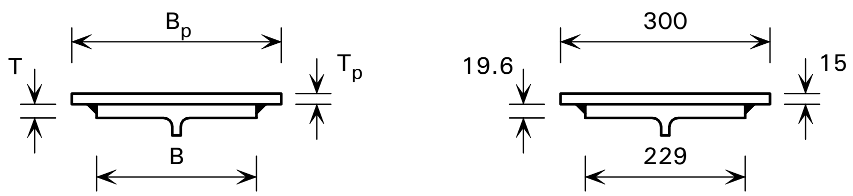
Grade of steel = S275 — 3.1.1

T > 16 mm and < 40 mm for the UB — Table 9

Therefore for classification take $p_y = 265$ N/mm^2 for both the UB and the plate

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{265}} = 1.02$$

Flange classification



Example 20 Gantry girder, using a UB with a plated top flange	Sheet 9 of 22	Rev

For a compound flange, the following width to thickness ratios should be considered: 3.5.3

(a) The ratio of the outstand of the compound flange to the thickness of the original flange should be classified under "outstand element of compression flange - rolled section".

For the outstand element of a compression flange of a rolled section, the limiting *b/T* for a class 1 plastic flange is $9\varepsilon = 9 \times 1.02 = 9.18$

$$\text{The actual } b/T = \frac{B_P/2}{T} = \frac{300/2}{19.6} = 7.7 < 9.18$$

(b) The ratio of the internal width of the plate between the lines of weld or bolts connecting it to the original flange, to the thickness of the plate should be classified under "internal element of compression flange".

For the internal element of a compression flange, the limiting *b/T* for a class 1 plastic flange is $28\varepsilon = 28 \times 1.02 = 28.6$

$$\frac{B}{T_p} = \frac{229}{15} = 15.3 < 28.6$$

(c) The ratio of the outstand of the plate beyond the lines of welds or bolts connecting it to the original flange, to the thickness T_p of the plate should be classified under "outstand element of compression flange - welded section".

For the outstand element of a compression flange of a welded section, the limit for a class 1 plastic flange is $8\varepsilon = 8 \times 1.02 = 8.16$

$$\frac{(B_P - B)/2}{T_p} = \frac{(300-229)/2}{15} = 2.4 < 8.16$$

Therefore, **the flange is class 1 plastic.**

Web classification

The equal area axis and the neutral axis of the compound section lie above the mid-depth of the UB section (see Figure 20.9) because the plate is connected to the top flange of the UB section. Before determining the value of y_{ea} and y_{na}, the web classification can be checked conservatively by assuming that the neutral axis is at the mid-depth of the UB Section.

The limiting *d/t* for a class 1 plastic web is
$80\varepsilon = 80 \times 1.02 = 81.6$ Table 11

$$\text{The actual } \frac{d}{t} = \frac{547.6}{11.9} = 46.0 < 81.6$$

Therefore, **the web is class 1 plastic.**

Flange and web are both class 1

Therefore, **the cross section is class 1, plastic**

20.3.2 Section properties

A summary of the section properties for the top plate, UB section and compound section (UB + top plate) are given in Table 20.1. The calculations are given in Section 20.5 below.

Table 20.1 *Section properties*

			Plate* 300 x 15	UB** 610x229x125	Compound section*** (UB + plate)
Area	A	cm^2	45	159	204
Second moment of area x-x	I_x	cm^4	8.44	98600	133100
Second moment of area y-y	I_y	cm^4	3375	3930	7305
Torsional constant	J	cm^4	33.8	154	187.8
Radius of gyration x-x	r_x	cm	-	24.9	-
Radius of gyration y-y	r_y	cm	-	4.97	5.98
Elastic modulus x-x	Z_x	cm^3	-	3220	Top 5284; Bottom 3547
Elastic modulus y-y	Z_y	cm^3	225	343	487
Plastic modulus x-x	S_x	cm^3	-	3680	4622
Plastic modulus y-y	S_y	cm^3	337.5	535	-
Warping constant	H	cm^6	-	3.45×10^6	-
Torsional index	x		-	34.1	35.3
Buckling parameter	u		-	0.874	0.86
Flange ratio$^+$	η		-	-	0.73

Only the properties required in the design are given in the table.
* For calculation of section properties of the plate, see Section 20.5.1.
** Section properties of UB taken from Volume 1[2]
*** For calculation of section properties of the compound section, see Section 20.5.3.
$^+$ Required for LTB check in section 20.4.2

20.4 Design Checks

20.4.1 Major axis bending

Basic requirement: $M_x \leq M_{cx}$ 4.2.5

The moment capacity for a class 1 plastic section is given by:

$M_{cx} = p_y S_x$

$M_{cx} = 265 \times 4622 \times 10^{-3} = 1225$ kNm

Check limit to avoid irreversible deformation under serviceability loads. 4.2.5.1

For a simply supported beam $M_{cx} \leq 1.2\, p_y Z_x$

Z_x is the lesser of $Z_{x,bottom}$ and $Z_{x,top}$

$Z_x = Z_{x,bottom} = 3547$ cm^3

$1.2 p_y Z_x = 1.2 \times 265 \times 3547 \times 10^{-3} = 1128$ kNm

Therefore $M_{cx} = 1128$ kNm

In crane load combination 1, $M_x = 375$ kNm (see Figure 20.4)

375 kNm < 1128 kNm

Therefore, **the major axis moment capacity is adequate.**

Example 20 Gantry girder, using a UB with a plated top flange	Sheet 11 of 22	Rev

20.4.2 Lateral-torsional buckling

Check gantry girder as an unrestrained member for vertical loads.

Due to interaction between crane wheels and crane rails, crane loads need not be treated as destabilizing, assuming that the rails are not mounted on resilient pads. 4.11.3

No account should be taken of the effect of moment gradient i.e. m_{LT} should be taken as 1.0. 4.11.3

Hence **the basic requirement is $M_x \leq M_b$** 4.3.6.3 and 4.3.6.2

The buckling resistance moment M_b for a class 1 plastic section is given by: 4.3.6.4

$M_b = p_b S_x$

where p_b is the bending strength and is dependent on the design strength p_y and the equivalent slenderness λ_{LT}.

$$\lambda_{LT} = uv\lambda\sqrt{\beta_W}$$ 4.3.6.7(a)

For a class 1 plastic section, $\beta_w = 1$ 4.3.6.9

The ends of the girder are torsionally restrained. At the supports the compression flanges are laterally restrained but both the flanges are free to rotate on plan. The effective length L_E for normal loading condition, is given as: 4.3.5.1 Table 13

$$L_E = 1.0L \quad = 8.0 \text{ m}$$

$$\lambda = \frac{L_E}{r_y} \quad = \frac{8.0}{5.98} \times 10^2 \quad = 133.8$$

$$\frac{\lambda}{x} = \frac{133.8}{35.3} \quad = 3.8$$

$$\eta = 0.73$$

For $\frac{\lambda}{x} = 3.8$ and $\eta = 0.73$, Table 19 gives, $v = 0.78$

$$\lambda_{LT} = 0.86 \times 0.78 \times 133.8 = 89.8$$

For p_y = 265 N/mm and λ_{LT} = 89.9, Table 17 gives p_b = 131 N/mm^2 Table 17

$$M_b = p_b S_x = 131 \times 4622 \times 10^{-3} = 605 \text{ kNm}$$

In crane load combination 1, M_x = 375 kNm (see Figure 20.4)

375 kNm < 605 kNm

Therefore, **the buckling resistance moment is adequate.**

20.4.3 Horizontal moment capacity

Horizontal loads are assumed to be carried by the top flange plate only.

Basic requirement: $M_R \leq M_{c,plate}$

Moment capacity of the top flange plate, $M_{c,plate}$ is equal to the lesser of $1.2p_y Z_{plate}$ and $p_y S_{plate}$. 4.2.5.1

$$Z_{plate} = \frac{15 \times 300^2}{6} \times 10^{-3} \quad = 225 \text{ cm}^3$$

Example 20 Gantry girder, using a UB with a plated top flange	Sheet 12 of 22	Rev

$$S_{\text{plate}} = \frac{15 \times 300^2}{4} \times 10^{-3} = 337.5 \text{ cm}^3$$

The design strength of the flange plate **for this check** can be taken as $p_y = 275 \text{ N/mm}^2$, since the plate thickness < 16 mm.

$$1.2 p_y Z_{\text{plate}} = 1.2 \times 275 \times 225 \times 10^{-3} = 74.2 \text{ kNm}$$

$$p_y S_{\text{plate}} = 275 \times 337.5 \times 10^{-3} = 92.8 \text{ kNm}$$

Therefore, $M_{\text{c,plate}} = 74.2$ kNm

Maximum horizontal moment is due to crabbing in crane load combination 3, $M_R = 26.0$ kNm (see Figure 20.8)

26.0 kNm < 74.2 kNm

Therefore, **the minor axis moment capacity is adequate.**

20.4.4 Consider combined vertical and horizontal moments

This check should be carried out using the loads calculated for crane load combination 3:

1.4 dead load + 1.4 vertical crane loads + 1.4 horizontal crane loads

Section capacity

Basic requirement: $\frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$ 4.8.3.2(a)

For M_x and M_y see Figures 20.6 and 20.8 respectively.
$M_{cy} = M_{\text{c,plate}}$

$$\frac{330}{1128} + \frac{26.0}{74.2} = 0.293 + 0.350 = 0.64 < 1$$

Therefore, **the section capacity is adequate.**

Buckling resistance

In the "simplified method", the following relationships must be satisfied: 4.8.3.3.1

$$\frac{m_x M_x}{p_y Z_x} + \frac{m_y M_y}{p_y Z_y} \leq 1 \quad \text{and}$$

$$\frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \leq 1$$

For simplicity take maximum M_x and M_y (rather than coexistant M_x and M_y) and assume that the minor axis loads are carried by the plate only.
M_{LT} is the maximum major axis moment in the segment = 330 kNm (see Figure 20.6).
M_y is the maximum crabbing moment = 26.0 kNm (see Figure 20.8)
m_{LT} is taken as 1.0. 4.11.3

For simplicity take $m_x = m_y = 1.0$.
First equation

$$\frac{330 \times 10^6}{265 \times 3547 \times 10^3} + \frac{26.0 \times 10^6}{275 \times 225 \times 10^3} = 0.351 + 0.420 = 0.77 < 1$$

Second equation

$$\frac{330}{605} + \frac{26.0 \times 10^6}{275 \times 225 \times 10^3} = 0.545 + 0.420 = 0.97 < 1$$

Therefore, **the buckling resistance is adequate.**

20.4.5 Web shear at supports

Basic requirement $F_v \leq P_v$

$P_v = 0.6 p_y A_v$

In this example, it would be reasonable to assume that the shear is resisted by the UB-section.

$\therefore A_v = tD$ (for rolled I-sections, load parallel to web) 4.2.3(a)

$A_v = 11.9 \times 612.2 = 7285$ mm²

$P_v = 0.6 \times 265 \times 7285 \times 10^{-3} = 1158$ kN

Maximum shear $F_{vmax} = 248$ kN (see Figure 20.5 for crane load combination 1)

Note: Since $F_{vmax} < 0.6 P_v$ no reduction in moment capacity due to shear.

248 kN < 1158 kN

Therefore, **the shear capacity of the web at the supports is adequate.**

20.4.6 Local compression under wheels 4.11.4

Basic requirement: $f_w \leq p_y$

Local compression on the web may be obtained by distributing the crane wheel load over a length x_R where

$x_R = 2(H_R + T)$ but $x_R \leq s_w$

Assume rail height $H_R = 100$ mm

Combined flange thickness $T = 15 + 19.6 = 34.6$ mm

$x_R = 2(100 + 34.6) = 269.2$ mm

s_w is the distance between adjacent wheels $= a_w = 4000$ mm

Stress on the web under the wheel f_w

$$f_w = \frac{W'_w}{x_R t} = \frac{158 \times 10^3}{269.2 \times 11.9} = 49.3 \text{ N/mm}^2$$

$W'_w = 158$ kN for crane load combination 1 (see Section 20.2.1)

49.3 N/mm² < 265 N/mm²

Therefore, **the resistance to local compression under the wheels is adequate.**

20.4.7 Web bearing and buckling under the wheel

This should be checked in accordance with the rules given in clauses 4.5.2 and 4.5.3 of BS 5950-1:2000[1]. 4.5.2 4.5.3

The procedure is similar to that given in worked Example 2 – Restrained Beam.

20.4.8 Deflection

Limiting vertical deflection due to static wheel loads δ_{vLim}

2.5.2
Table 8(c)

$$\delta_{\text{vLim}} = \frac{span}{600} = \frac{8000}{600} = 13.3 \text{ mm}$$

Limiting horizontal deflection (calculated on the top flange properties alone) due to crane surge δ_{hLim}

$$\delta_{\text{hLim}} = \frac{span}{500} = \frac{8000}{500} = 16.0 \text{ mm}$$

The deflection of gantry girders can be important and the exact calculations can be complex with a system of rolling loads. For two equal loads, however, a useful assumption is that the maximum deflection occurs at the centre of the span when the loads are positioned equidistant about the centre.

(1) Maximum vertical deflection due to static vertical loads

$$\delta_{\text{vmax}} = \frac{W_{\text{us}} L^3}{6 E I_{\text{x}}} \left(\frac{3}{8}\left(1 - \frac{a_{\text{w}}}{L}\right) - \left(\frac{L - a_{\text{w}}}{2L}\right)^3 \right)$$

Maximum, unfactored static load per wheel	W_{us}	= 76 kN (see sheet 2)
Modulus of elasticity	E	= 205 kN/mm^2
Second moment of area of gantry girder	I_{x}	= 133100 cm^4 (see Table 20.1)
Wheel spacing in end carriage	a_{w}	= 4.0 m
Span of gantry girder	L	= 8.0 m

$$\delta_{\text{vmax}} = \frac{76 \times 8^3}{6 \times 205 \times 133100} \left(\frac{3}{8}\left(1 - \frac{4}{8}\right) - \left(\frac{8 - 4}{2 \times 8}\right)^3 \right) \times 10^5 \; = 4.1 \text{ mm}$$

4.1 mm < 13.3 mm

Therefore, **the maximum vertical deflection is acceptable.**

(2) Maximum horizontal deflection due to crane surge

$$\delta_{\text{h.surge}} = \frac{W_{\text{H1}} L^3}{6 E I_{\text{yp}}} \left(\frac{3}{8}\left(1 - \frac{a_{\text{w}}}{L}\right) - \left(\frac{L - a_{\text{w}}}{2L}\right)^3 \right)$$

Maximum unfactored transverse surge load per wheel

W_{H1} = 3 kN (see Sheet 2)

Second moment of area of flange plate only

I_{yp} = 3375 cm^4 (see Table 20.1)

$$\delta_{\text{h.surge}} = \frac{3 \times 8^3}{6 \times 205 \times 3375} \left(\frac{3}{8}\left(1 - \frac{4}{8}\right) - \left(\frac{8 - 4}{2 \times 8}\right)^3 \right) \times 10^5 = 6.4 \text{ mm}$$

6.4 mm < 16.0 mm

Therefore, **the maximum horizontal deflection due to crane surge is acceptable.**

(3) Maximum horizontal deflection due to crabbing

Maximum horizontal deflection at the centre with wheel position as shown in Fig 20.8

$$\delta_{\text{h.crab}} = \frac{F_R L^3}{48 EI_{yp}}$$

Unfactored crabbing force per wheel

$F_R = 9.3$ kN (see sheet 2)

$$\delta_{\text{h.crab}} = \frac{9.3 \times 8^3 \times 10^5}{48 \times 205 \times 3375} = 14.3 \text{ mm}$$

14.3 mm < 16.0 mm

Therefore, **the maximum horizontal deflection due to crabbing is acceptable.**

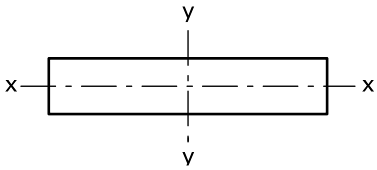
Adopt 610 x 229 x 125 UB grade S275 with a 300 mm x 15 mm plate in S275 steel.

20.5 Appendix: Section properties

20.5.1 Section properties of the plate

Width of plate $B_p = 300$ mm

Thickness of plate $T_p = 15$ mm



Cross-sectional area $A_p = 300 \times 15 \times 10^{-2}$ = 45 cm²

Second moment of area x-x $I_{xp} = \frac{300 \times 15^3}{12} \times 10^{-4}$ = 8.44 cm⁴

Second moment of area y-y $I_{yp} = \frac{15 \times 300^3}{12} \times 10^{-4}$ = 3375 cm⁴

Torsional constant $J_p = \frac{1}{3} B_p T_p^3 = 0.33 \times 300 \times 15^3 \times 10^{-4}$ = 33.8cm⁴

Elastic modulus y-y $Z_{yp} = \frac{I_{yp}}{B_p / 2} = \frac{3375}{15}$ = 225 cm³

Plastic modulus y-y $S_{yp} = \frac{15 \times 300^2}{4} \times 10^{-3}$ = 337.5 cm³

20.5.2 Section properties of the UB

From section property tables

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Area	A_B	=	159 cm^2
Second moment of area x-x	I_{xB}	=	98600 cm^4
Second moment of area y-y	I_{yB}	=	3930 cm^4
Torsion constant	J_B	=	154 cm^4

20.5.3 Section properties of compound section (UB + Plate)

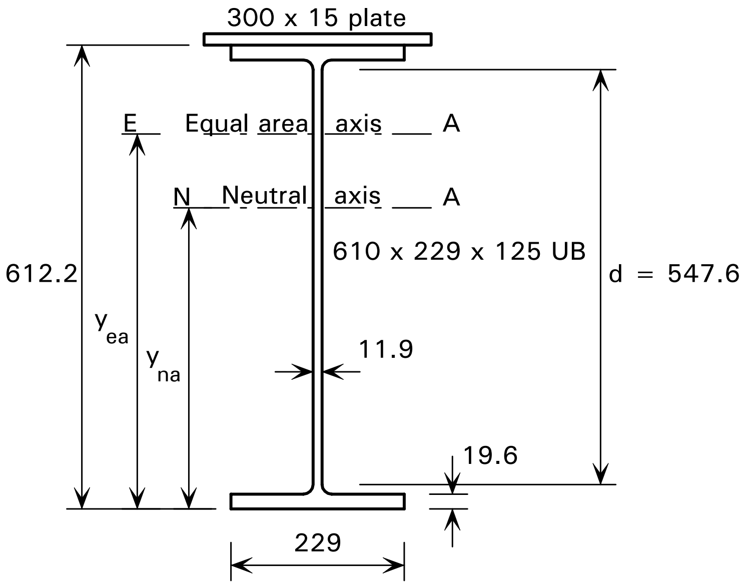

Figure 20.11 *Trial gantry girder section-UB with plate*

Elastic section properties

Neutral axis above the bottom flange, y_{na}

$$y_{na} = \frac{A_B \frac{D}{2} + A_p\left(D + \frac{T_p}{2}\right)}{A_B + A_p} = \frac{159\frac{612.2}{2} + 45 \times \left(612.2 + \frac{15}{2}\right)}{159 + 45} = 375.3\,\text{mm}$$

$$I_x = I_{xB} + A_B\left(y_{na} - \frac{D}{2}\right)^2 + I_{xp} + A_p\left(D - y_{na} + \frac{T_p}{2}\right)^2$$

$$= 98600 + 159 \times \left(375.3 - \frac{612.2}{2}\right)^2 \times 10^{-2} + 8.44 + 45\left(612.2 - 375.3 + \frac{15}{2}\right)^2 \times 10^{-2}$$

$$= 133100\,\text{cm}^4$$

$$Z_{xTop} = \frac{I_x}{D + T_p - y_{na}} = \frac{133101}{612.2 + 15 - 375.3} \times 10 = 5284\,\text{cm}^3$$

$$Z_{xBot} = \frac{I_x}{y_{na}} = \frac{133101}{375.3} \times 10 = 3547\ \text{cm}^3$$

$$I_y = I_{yB} + I_{yp}$$

$$I_{yp} = \frac{T_p B_p{}^3}{12} = \frac{15 \times 300^3}{12} \times 10^{-4} = 3375\ \text{cm}^4$$

$$I_y = 3930 + 3375 = 7305\ \text{cm}^4$$

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$$r_y = \left(\frac{I_y}{A}\right)^{0.5} = \left(\frac{7305}{204}\right)^{0.5} = 5.98 \text{ cm}$$

$$Z_y = \frac{I_y}{B_p/2} = \frac{7305}{300/2} \times 10 = 487 \text{ cm}^3$$

Plastic section properties

Assume that the UB and the plate have the same design strength.

Equal area axis above the bottom flange y_{ea}

Assuming E-A lies in the web then

$$y_{ea} = \frac{D}{2} + \frac{A_p}{2t} = \frac{612.2}{2} + \frac{45 \times 10^2}{2 \times 11.9} = 495.2 \text{ mm}$$

Ignoring the fillets, the plastic modulus of the combined section, S_x

$$S_x = BT\left(D - y_{ea} - \frac{T}{2}\right) + t\frac{(D - y_{ea} - T)^2}{2}$$
$$+ BT\left(y_{ea} - \frac{T}{2}\right) + t\frac{(y_{ea} - T)^2}{2}$$
$$+ A_p\left(D - y_{ea} + \frac{T_p}{2}\right) \times 10^2$$

$$= 229 \times 19.6\left(612.2 - 495.2 - \frac{19.6}{2}\right) + 11.9\frac{(612.2 - 495.2 - 19.6)^2}{2}$$
$$+ 229 \times 19.6\left(495.2 - \frac{19.6}{2}\right) + 11.9\frac{(495.2 - 19.6)^2}{2}$$
$$+ 45 \times 10^2 \times \left(612.2 - 495.2 + \frac{15}{2}\right)$$
$$= 4622000 \text{ mm}^3 = 4622 \text{ cm}^3$$

Torsion constant, J

$$J = J_B + J_p = 154 + 33.8 = 187.8 \text{ cm}^4$$

Torsional index, x

$$x = 0.566 h_s \left(\frac{A}{J}\right)^{0.5}$$ B.2.4.1

$$h_s = D + T_p - \frac{T}{2} - \left(\frac{\frac{B_p T_p^2}{2} + BT\left(T_p + \frac{T}{2}\right)}{B_p T_p + BT}\right)$$ B.2.4.1

$$= \left(612.2 + 15 - \frac{19.6}{2} - \left(\frac{\frac{300 \times 15^2}{2} + 229 \times 19.6 \left(15 + \frac{19.6}{2} \right)}{300 \times 15 + 229 \times 19.6} \right) \right) \times 10^{-1} \quad = 59.8 \text{ cm}$$

$$A \quad = A_B + A_p \quad = 159 + 45 \quad = 204 \text{ cm}^2$$

$$x \quad = 0.566 \times 59.8 \left(\frac{204}{187.8} \right)^{0.5} \quad = 35.3$$

Buckling parameter, u

B.2.4.1 and B.2.3

$$u \quad = \left(\frac{4 S_x{}^2 \gamma}{A^2 h_s{}^2} \right)^{0.25}$$

$$\gamma \quad = 1 - \frac{I_y}{I_x} \quad = 1 - \frac{7305}{133100} \quad = 0.95$$

$$u \quad = \left(\frac{4 \times 4622^2 \times 0.95}{204^2 \times 59.8^2} \right)^{0.25} \quad = 0.86$$

Flange ratio, η

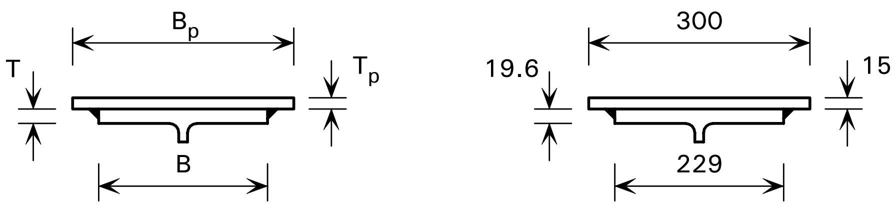

Figure 20.12 *Dimensions of compound flange*

B.2.4.1

$$\eta \quad = \frac{I_{yc}}{I_{yc} + I_{yt}}$$

I_{yc} is the second moment of area of compression flange about minor axis of the section.

$$I_{yc} \quad = \left(\frac{T_p B_p{}^3}{12} + \frac{TB^3}{12} \right) \quad = \left(\frac{15 \times 300^3}{12} + \frac{19.6 \times 229^3}{12} \right) \times 10^{-4} \quad = 5337 \text{ cm}^4$$

I_{yt} is the second moment of area of the tension flange about minor axis of the section.

$$I_{yt} \quad = \frac{TB^3}{12} \quad = \left(\frac{19.6 \times 229^3}{12} \right) \times 10^{-4} \quad = 1961 \text{ cm}^4$$

$$\eta \quad = \frac{5337}{5337 + 1961} \quad = 0.73$$

20.6 Appendix: Crane gantry girder loading

20.6.1 General

The loads to be used for the design of crane gantry girders are not very clearly defined by current British Standards. This section discusses the provisions of BS 5950-1:2000[1] and proposes a simplified loading to achieve a reasonably economical structure through simple design. BS 5950-1 makes reference to BS 2573-1:1983, *Rules for the design of cranes, Part 1, Specification for classification, stress calculations and design criteria for structures*[4]. In particular, BS 5950-1 uses the BS 2573 crane classifications Q1, Q2, Q3 and Q4. The descriptive definitions given in BS 2573 are as follows :

Table 20.2

Class	Descriptive definition
Q1	Cranes which hoist the safe working load very rarely and, normally, light loads.
Q2	Cranes which hoist the safe working load fairly frequently and, normally, moderate loads.
Q3	Cranes which hoist the safe working load fairly frequently and, normally, heavy loads.
Q4	Cranes which are normally loaded close to safe working load.

20.6.2 Loading in BS 5950-1:2000

The principal clauses on loading from cranes in BS 5950-1:2000 are:

Clause 2.4.1.3, Overhead travelling cranes:
"The γ_f factors given in Table 2 (Partial factors for loads γ_f) for vertical loads from overhead travelling cranes should be applied to the dynamic vertical wheel loads, ie the static vertical wheel loads increased by the appropriate allowance for dynamic effects."

Clause 2.2.3, Loads from overhead travelling cranes:
"For overhead travelling cranes, the vertical and horizontal dynamic loads and impact effects should be determined in accordance with BS 2573-1. The values for cranes of loading class Q3 and Q4 as defined in BS 2573-1 should be established in consultation with the crane manufacturer."

Clause 4.11.2, Crabbing of trolley
This clause states that gantry girders intended to carry cranes of loading class Q1 and Q2 as defined in BS 2573 need not be designed for crabbing loads. For girders intended to carry cranes of loading class Q3 and Q4 as defined in BS 2573, the design crabbing forces are defined.

20.6.3 Loading in BS 2573-1:1983

Vertical loads in BS 2573-1

BS 2573-1 increases the loads on the hook by a dynamic factor, but does not apply any dynamic factor to the self-weight of the crane. However, it does reduce the allowable design stresses by a "duty factor" to account for effects not considered in the analysis. It is not practical to transfer the entire design procedure of BS 2573 to BS 5950 because BS 2573 is a permissible stress code whereas BS 5950 is a limit state code.

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Horizontal loads in BS 2573-1

BS 2573-1 covers horizontal loads in clause 3.1.5. This is divided into sub-clauses:

> 3.1.5.1, *Inertia forces*, which gives no standard design force but makes it clear that it depends on the drive and brakes of each crane. This inertia force is commonly referred to as the surge load.
>
> 3.1.5.2, *Skew loads due to travelling*, which gives design loads, but BS 5950-1:2000 defines the loads to be used in design of the girder in clause 4.11.2. This is commonly referred to as the crabbing force.
>
> 3.1.5.3, *Buffer loads*, which gives no standard design force but makes it clear that it depends on each crane and gives some design guidance.

From above, it is clear that the best possible information should be sought on the design of the particular crane.

Traditionally, horizontal loads were taken as a transverse load of 10% of the static vertical reactions and a longitudinal load of 5% of the static vertical reactions, but not acting at the same time. These are the loads given in BS 449[5] and were included in BS 6399-1:1984[6], but do not appear in BS 6399-1:1996[7]. One of the reasons for removing these factors is that they under-estimate the forces exerted by some modern cranes. There is anecdotal evidence of rare cases where the horizontal force can be as high as 20% to 30% of the vertical loads.

20.6.4 Concern about the use of BS 2573 impact factor alone for vertical load

BS 2573 is a code for the design of cranes, not of crane gantry girders. As a crane moves along the rails, it will pass over irregularities such as joints in the rails. These will cause dynamic loads on the girder in addition to the static loads. This is a load case that must be considered. If a crane has a high self-weight but only a relatively light lifted load, design loads derived from BS 2573 might underestimate the vertical loads applied to the crane girder. Therefore, the designer should consider whether the dynamic effects of the crane plus lifted load moving along the rail could give a worse vertical load than when the crane is stationary and lifting its load. Where the lifting case clearly gives the worst vertical load, loads from the crane moving need not be calculated. Where the lifting case does not clearly give the worst vertical load, loads from the crane moving should also be calculated as a separate vertical load case. In the absence of better information, it is prudent to use the traditional dynamic factors that have been used for decades to cover any cases that might be omitted by the use of BS 2573.

Traditionally, crane gantry girder design has allowed for dynamic effects by using the values in BS 449, giving an additional 25% on static vertical reactions from the total crane self-weight plus lifted load. Where better information is impossible to obtain at the time of design, these traditional factors may be used in addition to the case using BS 2573 as a reasonable basis of design for any load cases that might be ignored by the use of the BS 2573 impact factor on the lifted load alone.

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The recommended vertical load cases for the gantry girder can be summarised as follows:

Table 20.3 *Factored vertical loads*

Vertical load cases	Total wheel load on rail
Crane stationary, lifting load	$F \times \gamma_f \times R_h + \gamma_f \times R_s$
Crane moving with load	$1.25 \times \gamma_f \times R_h + 1.25 \times \gamma_f \times R_s$

Where:
F is the impact factor from BS 2573-1
γ_f is the partial factor for vertical crane loads from BS 5950-1:2000
R_h is the unfactored vertical wheel reaction from the hook load
R_s is the unfactored vertical wheel reaction from the crane self weight

20.6.5 Loading for this example

This example illustrates design of the crane gantry girders for one of the most common types of crane ratings. This an overhead travelling crane with a vertical impact factor, according to BS 2573-1, of 1.3 This factor appears in BS 2731-1 for medium and heavy duty in warehouses and workshops in Table 4(a) *Overhead travelling industrial type cranes (O.T.C.)* and for medium duty (general use) in Table 4(c) *Transporters*

The purpose of this example is to present the design procedure for crane girders so that a reasonable economic design is achieved with reasonable economy of design effort. Because this example is not done by computer, it is important to reduce the number of load cases to as few as possible.

Vertical loads
Where the BS 2573 impact factor is 1.3, the loading for the two load cases from Table 20.3 may be re-summarised as:

Table 20.4 *Factored vertical loads*

Vertical load cases	Total wheel load on rail
Crane stationary, lifting load	$1.3 \times \gamma_f \times R_h + \gamma_f \times R_s$
Crane moving with load	$1.25 \times \gamma_f \times R_h + 1.25 \times \gamma_f \times R_s$

The traditional factor of 1.25 is only slightly less than BS 2753 factor of 1.3. Therefore, to minimise the number of load cases and simplify the calculations as far as possible, this example will use a factor of 1.3 applied simultaneously to both the lifted load and to the self-weight of the crane. This is an envelope case which preserves a balance between economy of structure and economy of design effort while ensuring safety.

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Table 20.5 *Factored vertical wheel load used in Example 20*

Vertical load cases	**Total wheel load on rail**
Envelope case	$1.3 \times \gamma_f \times R$ $(= 1.3 \times \gamma_f \times R_h + 1.3 \times \gamma_f \times R_s)$
Where: R = is the unfactored vertical wheel reaction from the hook load plus crane self weight (crane bridge + crab)	

Horizontal loads

The horizontal loads due to surge or inertia forces are taken as:

1) transverse load of 10% of the combined weight of the crab and the lifted load
2) longitudinal load of 5% of the static vertical reactions (i.e. from the weight of the crab, crane bridge and lifted load).

Crabbing forces acting transverse to the rail are obtained from clause 4.11.2 (BS 5950-1:2000) because the crane is class Q3. If the crane were class Q1 or Q2, then the crabbing forces would not need to be considered.

The Steel Construction Institute Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 623345 Fax: (01344) 622944 **CALCULATION SHEET**	Job No. CDS 153		Sheet 1 of 13	Rev
	Job Title Example no. 21			
	Subject Simply supported stiffened plate girder			
	Client SCI	Made by JBL	Date Jun 2003	
		Checked by ASM	Date Oct 2003	

21 Simply supported stiffened plate girder

21.1 Introduction

The girder shown in Figure 21.1 is laterally restrained throughout its length. For the loading shown, design a stiffened plate girder in S275 steel. Girder depth unrestricted.

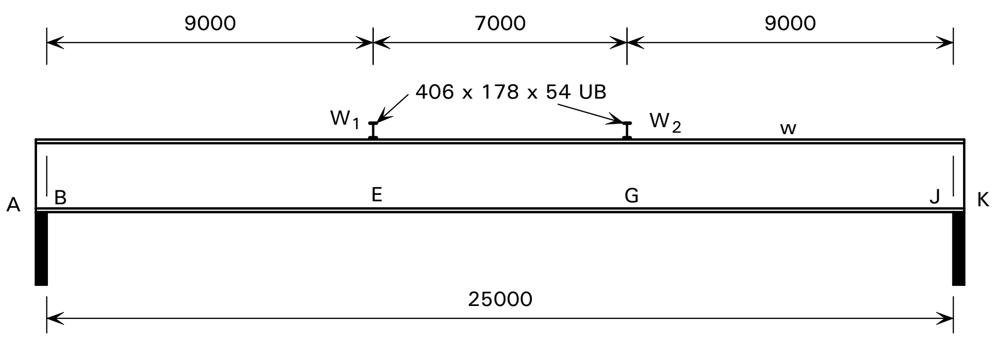


Figure 21.1

21.1.1 Loading (unfactored)

Dead loads:

UDL	w_d	= 20 kN/m
Point load	W_{1d}	= 200 kN
Point load	W_{2d}	= 200 kN

Imposed loads:

UDL	w_i	= 40 kN/m
Point load	W_{1i}	= 300 kN
Point load	W_{2i}	= 300 kN

21.1.2 Load Factors

Table 2
2.4.1.1

Dead load factor	γ_{fd}	= 1.4
Imposed load factor	γ_{fi}	= 1.6

21.1.3 Factored loads

$w' = w_d\,\gamma_{fd} + w_i\,\gamma_{fi} = (20 \times 1.4) + (40 \times 1.6) = 92$ kN/m

$W_1' = W_{1d}\,\gamma_{fd} + W_{1i}\,\gamma_{fi} = (200 \times 1.4) + (300 \times 1.6) = 760$ kN

$W_2' = W_{2d}\,\gamma_{fd} + W_{2i}\,\gamma_{fi} = (200 \times 1.4) + (300 \times 1.6) = 760$ kN

The design shear forces and moments are as shown in Figure 21.2.

21.1.4 Initial sizing of plate girder

When there is no depth restriction, plate girders usually vary in depth between 1/8 of the span for short girders and 1/15 of the span for long girders. Here the depth will be assumed to be approximately 1/12 × span.

Depth, $D \approx$ span/12   Depth $\approx 25000/12 \approx 2000$ mm

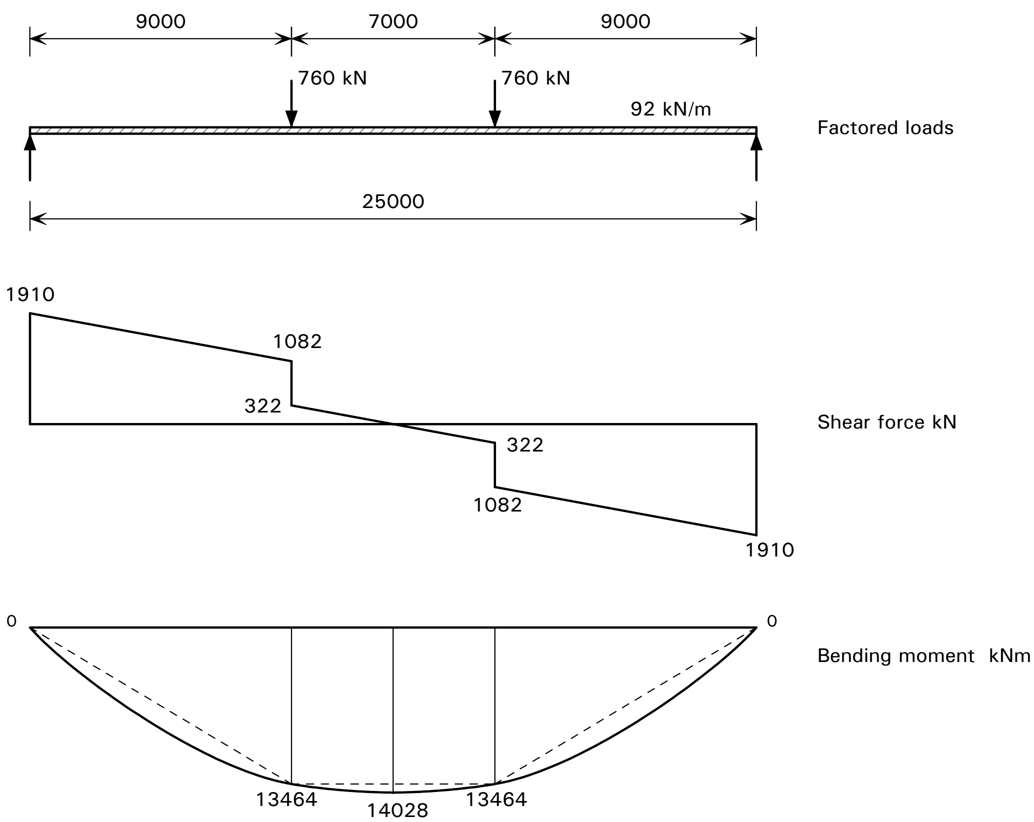


Figure 21.2 *Design loading, shear and moment diagram*

21.2 Member checks

21.2.1 Choose a suitable girder size

The initial trial section size is selected to give a suitable moment capacity.

Assume flange thickness between 16 mm and 40 mm, hence $p_y = 265$ N/mm^2

$A_f \approx M_{max} / Dp_y \quad A_f \approx \dfrac{14028}{2000 \times 265} \times 10^6 \approx 26500$ mm^2

Choice of web thickness is generally a matter of experience and is here taken as depth/150 approximately. $t \approx$ Depth/150 $t \approx 2000/150 \approx 13$ mm

Try girder with flanges 700 mm x 40 mm and web 1950 mm x 13 mm

Example 21 Simply supported stiffened plate girder	Sheet 3 of 13	Rev

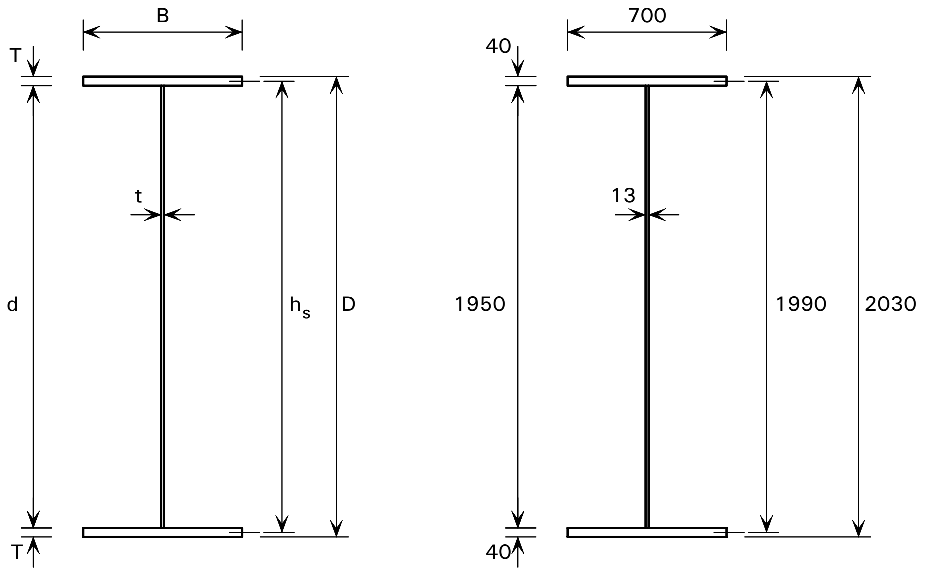


Figure 21.3 *Plate girder dimensions*

21.2.2 Section classification

Flange

Grade of steel = S275 — 3.1.1 Table 9

16 mm < $T \leq 40$ mm

Therefore p_{yf} = 265 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_{yf}}} = \sqrt{\frac{275}{265}} = 1.02$$

Outstand, $b = \dfrac{B-t}{2} = \dfrac{700-13}{2} = 343.5$ mm — Figure 5

$$\frac{b}{T} = \frac{343.5}{40} = 8.59$$

For the outstand element of a compression flange, the limiting b/T for a welded class 2 compact flange is 9ε. — 3.5.2 Table 11

Limiting b/T = 9ε = 9.18

The actual b/T = 8.59 < 9.18

Therefore, **the flange is class 2 compact**

Web

Grade of steel = S275 — 3.1.1 Table 9

$t \leq 16$ mm

Therefore p_{yw} = 275 N/mm^2

$$\varepsilon = \sqrt{\frac{275}{p_{yw}}} = \sqrt{\frac{275}{275}} = 1.0$$

d/t = 1950/13 = 150

Since the section is symmetrical and subject to pure bending, the neutral axis is at mid-depth. For this case, the limiting d/t for a class 3 semi-compact web is 120ε. — 3.5.2 Table 11

Limiting d/t = 120ε = 120.0

The actual d/t = 150 > 120.0

Therefore, **the web is class 4 slender**

21.2.3 Check for susceptibility to shear buckling

If the web depth-to-thickness ratio d/t of a plate girder exceeds 62ε, the web is susceptible to shear buckling. 4.4.4.2

Limiting d/t $= 62\varepsilon$ $= 62.0$
The actual d/t $= 150 > 62.0$
Therefore, **check the web for shear buckling** 4.4.5.1

21.2.4 Dimensions of web and flanges

4.4.3

For webs with transverse stiffeners, assuming stiffener spacing $a > d$, the minimum web thickness t to avoid serviceability problems is $d/250$ 4.4.3.2(b)

Minimum t $= d/250 = 1950/250$ $= 7.8$ mm
The actual t $= 13$ mm > 7.8 mm
Therefore, **web thickness is adequate for serviceability**

For webs with intermediate transverse stiffeners, assuming stiffener spacing $a > 1.5d$, the minimum web thickness to avoid compression flange buckling into the web is $(d/250)(p_{yf}/345)$ 4.4.3.3(b)

Minimum t $= (d/250)(p_{yf}/345)$ $= (1950/250)(265/345)$ $= 6.0$ mm
The actual $t = 13$ mm > 6.0 mm
Therefore, **web is adequate to avoid flange buckling into the web**

Note: For hybrid girders, i.e. those using higher grade steel for the flanges than for the web, due account should be taken of the variation in design strength between the component parts, but the classification of the web should be based on p_{yf} and not p_{yw}. 4.4.2 Note

However, in this example p_{yf} $= 265$ N/mm^2
and p_{yw} $= 275$ N/mm^2

21.2.5 Check the moment capacity

For welded sections with webs susceptible to shear buckling (i.e. $d/t > 62\varepsilon$), three methods for calculating the moment capacity M_c of the cross-section are given in the code. In this example the “flanges only” method of clause 4.4.4.2(b) is used. In this method a conservative value of M_f for the moment capacity is obtained by assuming that the moment is resisted by the flanges only, with each flange subject to uniform stress not exceeding p_{yf}. 4.4.4.2

Basic requirement $M_x \leq M_c$
M_c $= M_f = p_{fy} A_f h_s$
p_{fy} $= 265$ N/mm^2
A_f $= BT = 700 \times 40 = 28000$ mm^2
h_s $=$ distance between centroids $= 1990$ mm
M_f $= 265 \times 28000 \times 1990 \times 10^{-6} = 14766$ kNm
$\therefore$ $M_c = 14766$ kNm
M_x $= 14028$ kNm
14028 kNm $<$ 14766 kNm
Therefore, **the moment capacity is adequate.**

Example 21 Simply supported stiffened plate girder	Sheet 5 of 13	Rev

21.3 Panel checks

For webs with intermediate transverse stiffeners, three methods are given for checking the webs. In this example the simplified method of clause 4.4.5.2 is used. 4.4.5.1
Try the stiffener spacing shown in Figure 21.4.

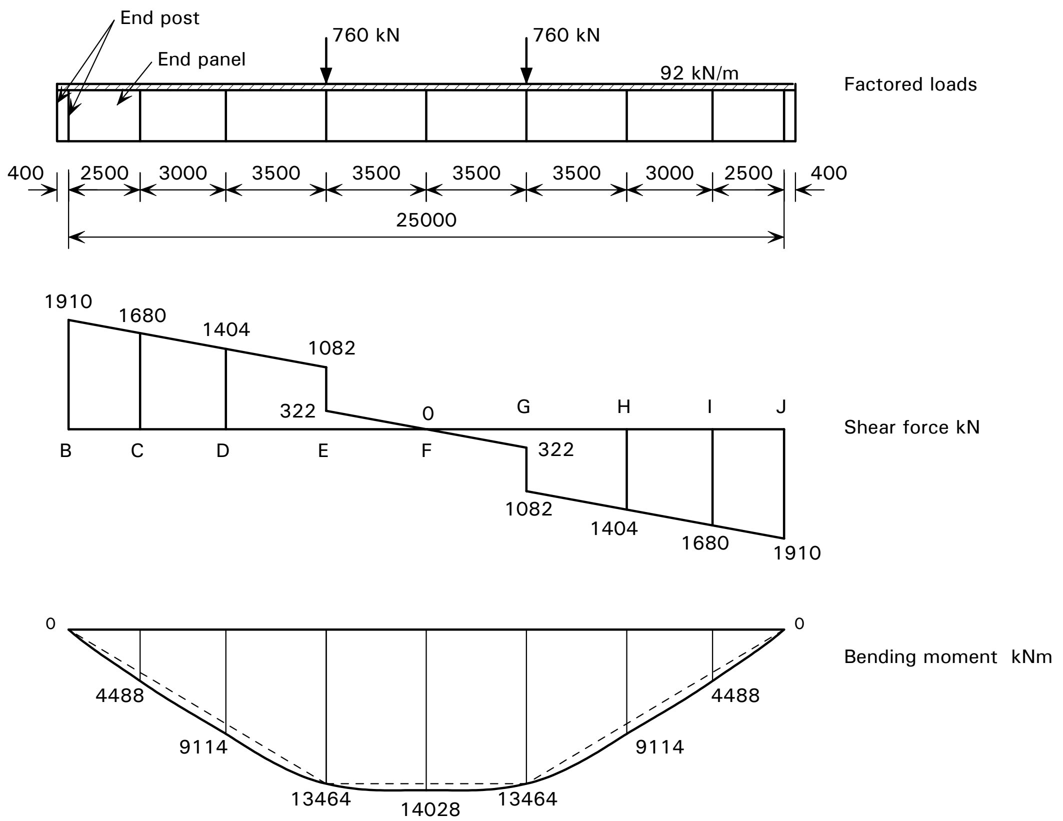


Figure 21.4 *Stiffener spacing, shear and moment diagram*

21.3.1 Check end panel BC

Basic requirement $F_v < V_b$ 4.4.5.2

From shear force diagram (Figure 21.4), maximum shear force in the panel:

$F_v = 1910\text{kN}$

$V_b = V_w = dtq_w$

$d/t = 1950/13 = 150$

$a/d = 2500/1950 = 1.28$

From Table 21, $q_w = 93.4$ N/mm^2 (for $p_y = 275$ N/mm^2) Table 21

$\therefore V_b = dtq_w = 1950 \times 13 \times 93.4 \times 10^{-3} = 2368$ kN

1910 kN < 2368 kN

Therefore, **the shear capacity of panel BC is adequate.**

21.3.2 Check panels CD and DE

Although the maximum shear forces on panels CD and DE are less than that on panel BC, both panels should be checked for shear due to increased stiffener spacing, a. However, calculations (not included here) will show that both panels CD and DE are satisfactory in this respect.

21.3.3 Determine whether end anchorage to panel BC is required

End anchorage need not be provided if either of the following conditions apply:

Condition 1: $V_w = P_v$ indicating that the shear capacity is the governing criteria — 4.4.5.4(a)

$V_w = dtq_w = 2368$ kN (from 21.3.1 above) — 4.4.5.2

$A_v = td = 13 \times 1950 = 25350\ \text{mm}^2$

$P_v = 0.6 p_{yw} A_v = 0.6 \times 275 \times 25350 \times 10^{-3} = 4183$ kN — 4.2.3(b)

2368 kN $\neq$ 4183 kN

Therefore **condition 1 does not apply**

Condition 2: $F_v \leq V_{cr}$ indicating that sufficient shear buckling resistance is available without forming a tension field — 4.4.5.4(b)

If $V_w \leq 0.72\ P_v$ then $V_{cr} = (V_w/0.9)^2/P_v$

$0.72 P_v = 0.72 \times 4183$ kN $= 3012$ kN

A_s $V_w \leq 0.72\ P_v$ (i.e. 2368 kN $\leq$ 3012 kN)

$$\therefore V_{cr} = \frac{\left(\frac{V_w}{0.9}\right)^2}{P_v} = \frac{\left(\frac{2368}{0.9}\right)^2}{4183} = 1655 \text{ kN}$$

At the end of the panel, $F_v = 1910$ kN

1910 kN > 1655 kN

Therefore **condition 2 does not apply**

Neither condition 1 nor 2 apply therefore end anchorage should be provided.

21.3.4 Check end post AB (providing end anchorage to panel BC) — H.4

Conservatively, anchor force $H_q = 0.5\, dtp_y \left(1 - \frac{V_{cr}}{P_v}\right)^{0.5}$ — H.4.1

or since $F_v < V_w$ (i.e 1910 kN < 2368 kN) the anchor force is given by:

$$H_q = 0.5\, dtp_y \left(\frac{F_v - V_{cr}}{V_w - V_{cr}}\right)\left(1 - \frac{V_{cr}}{P_v}\right)^{0.5}$$

$$= 0.5 \times 1950 \times 13 \times 275 \times 10^{-3} \left(\frac{1910 - 1655}{2368 - 1655}\right)\left(1 - \frac{1655}{4183}\right)^{0.5} = 970 \text{ kN}$$

End anchorage will be provided by a twin stiffener end post AB as shown in Figure 21.5 below. — H.4.3

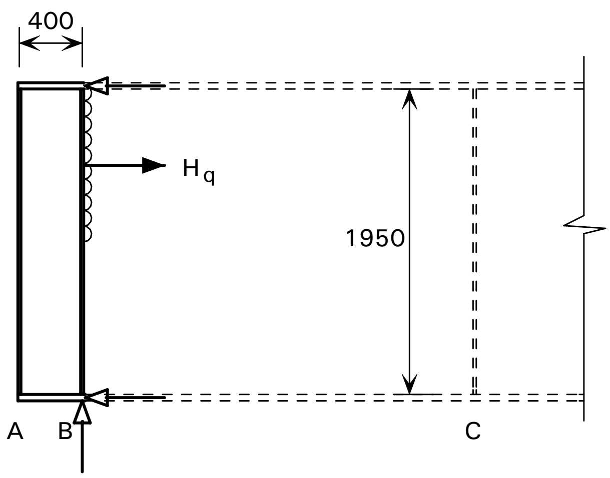


Figure 21.5 *Twin stiffener end post AB*

Example 21 Simply supported stiffened plate girder	Sheet 7 of 13	Rev

Basic requirement $R_{ft} \leq V_{cr.ep}$ — H.4.3

$R_{tf} = 0.75H_q = 0.75 \times 970 = 728$ kN

$V_{cr.ep}$ is the critical shear buckling resistance (see clause 4.4.5.4) of the web of the end post, treated as a **beam spanning between the flanges** of the girder.

$P_{v.ep} = 0.6 p_{yw} A_{v.ep} = 0.6 \times 275 \times 400 \times 13 \times 10^{-3} = 858$ kN — 4.2.3

Using the simplified method, $V_{w.ep} = d_{ep}\, t q_w$ — 4.4.5.2

For $a_{ep}/d_{ep} = 1950/400 = 4.9$ and $d_{ep}/t = 400/13 = 30.8$

Table 21 gives $q_w = 165$ N/mm^2 — Table 21

$\therefore V_{w.ep} = 400 \times 13 \times 165 \times 10^{-3} = 858$ kN

Since $V_{w.ep} = P_{v.ep}$ then $V_{cr.ep} = P_{v.ep}$

$\therefore V_{cr.ep} = 858$ kN

$R_{tf} = 728$ kN < 858 kN

Therefore, **the capacity of the end post is adequate.**

The end stiffener at A should be designed to resist the following compressive force — H.4.3

$$F_{tf} = 0.15 H_q \left(\frac{d}{a_e}\right) = 0.15 \times 970 \times \left(\frac{1950}{400}\right) = 710 \text{ kN}$$

Since $F_{tf} < F_s$ (710 kN $<$ 1910 kN) the stiffener at B should be designed to resist only the support reaction (1910 kN) of the girder.

21.4 Check stiffener at B as bearing stiffener (cl.4.5.2) and load carrying stiffener (cl.4.5.3)

21.4.1 Choose a suitable stiffener size

The stiffener may be designed to resist only the support reaction of the girder (see above).

Design force due to bearing

$F_x = F_v = 1910$ kN

Try stiffener 2 flats 200 mm x 16 mm grade S275

The effective load carrying stiffener is as shown in Figure 21.6 — 4.5.3.3

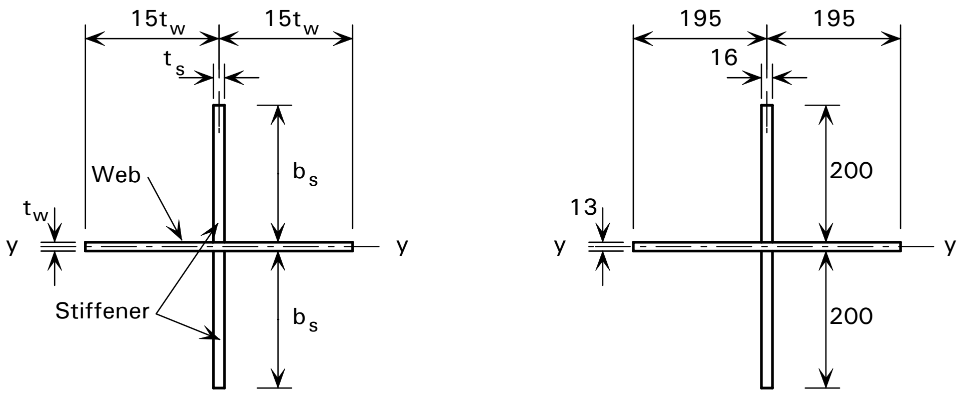


Figure 21.6 *Stiffened web at B*

Grade of steel = S275

$t_s \leq 16$ mm

Therefore $p_y = 275$ N/mm^2

Example 21 Simply supported stiffened plate girder	Sheet 8 of 13	Rev

The outstand of the web stiffener should not exceed $19\varepsilon t_s$ from the face of the web. 4.5.1.2

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{275}} = 1$$

Limiting outstand is $19\varepsilon t_s$ = 19 ×1 × 16 = 304 mm
Actual outstand is 200 mm

200 mm < 304 mm
Therefore, **the outstand is acceptable**

If the outstand of a stiffener is between $13\varepsilon t_s$ and $19\varepsilon t_s$ then its design should be based on an effective cross-section with an outstand of $13\varepsilon t_s$ 4.5.1.2

Limiting length is $13\varepsilon t_s$ = 13 ×1 × 16 = 208 mm
Actual outstand is 200 mm

200 mm < 208 mm
Therefore, **the outstand is fully effective**

21.4.2 Check stiffened web

Basic requirement: $(F_x - P_{bw}) \le P_s$ 4.5.2.2

$P_s = A_{s.net} p_y$

Allowing 12 mm cope for web/flange weld

$A_{s.net}$ = 2 × 188 × 16 = 6016 mm^2

P_s = 6016 × 275 × 10^{-3} = 1654 kN

$P_{bw} = (b_1 + nk)tp_{yw}$ 4.5.2.1

Assume stiff bearing length b_1 = 0

n = 5 as not at end of member

k = T = 40 mm

P_{bw} = (0 + 5 × 40) × (13 × 275) × 10^{-3} = 715 kN

$F_x - P_{bw}$ = 1910 - 715 = 1195 kN
1195 kN < 1654 kN
Therefore, **the bearing capacity of the stiffened web is adequate.**

21.4.3 Check buckling resistance of load carrying stiffener

Basic requirement: $F_x \le P_x$ 4.5.3.3

Referring to Figure 21.6

$$I_y = \frac{16 \times 413^3}{12} + \frac{390 \times 13^3}{12} - \frac{16 \times 13^3}{12} = 9400 \times 10^4 \text{ mm}^4$$

Effective area of the cruciform section.

$$A_s = 2 \times 200 \times 16 + 2 \times 195 \times 13 = 11470 \text{ mm}^2$$

$$r_y = \left(\frac{I_y}{A_s}\right)^{0.5} = \left(\frac{9400 \times 10^4}{11470}\right)^{0.5} = 90.5 \text{ mm}$$

$$L_E = 0.7L = 0.7 \times 1950 = 1365 \text{ mm}$$ 4.5.3.3

$$\lambda = \frac{L_E}{r_y} = \frac{1365}{90.5} = 15.1$$

From Table 24, p_c = 275 N/mm^2 Table 24 (strut curve c)

$P_x = A_s p_c$ = 11470 × 275 ×10^{-3} = 3154 kN

F_x = 1910 kN < 3154 kN

Therefore, **the buckling resistance of the stiffener is adequate.**

Example 21 Simply supported stiffened plate girder	Sheet 9 of 13	Rev

21.5 End stiffener at A

The force required to be resisted by the end stiffener at A is equal to F_{tf} due to anchor (calculated in Section 21.3.4): H.4.3

$F_A = F_{tf} = 710$ kN

By inspection, the same size stiffeners as at B (2 flats 200 mm × 16 mm S275) will be adequate.

21.6 Intermediate stiffener at C 4.4.6

21.6.1 Choose a suitable stiffener size

The minimum stiffness of the intermediate stiffener at C should satisfy both web panels either side of the stiffener: web panels BC and CD

Panel BC

$$\frac{a}{d} = \frac{2500}{1950} = 1.28$$

$1.28 < \sqrt{2}$

∴ Minimum second moment of area $I_s = 1.5(d/a)^2 d t_{min}^3$ 4.4.6.4

Conservatively, t_{min} is taken as the actual web thickness rather than the minimum required.

$t_{min} = 13$ mm

$I_s = 1.5 \times (1950/2500)^2 \times 1950 \times 13^3 = 391 \times 10^4$ mm^4

Panel CD

$$\frac{a}{d} = \frac{3000}{1950} = 1.54$$

$1.54 \geq \sqrt{2}$

∴ Minimum second moment of area $I_s = 0.75 d t_{min}^3$ 4.4.6.4

$t_{min} = 13$ mm

$I_s = 0.75 \times 1950 \times 13^3 = 321 \times 10^4$ mm^4

∴ Basic requirement: $I_s > 391 \times 10^4$ mm^4
Try stiffener 2 flats 100 mm x 13 mm grade S275

For stiffeners only

$$I_s = \left(\frac{13 \times 213^3}{12} - \frac{13 \times 13^3}{12}\right) = 1047 \times 10^4 \text{ mm}^4$$

1047×10^4 mm^4 > 391×10^4 mm^4
Therefore, **the stiffness of the stiffener at C is adequate.**

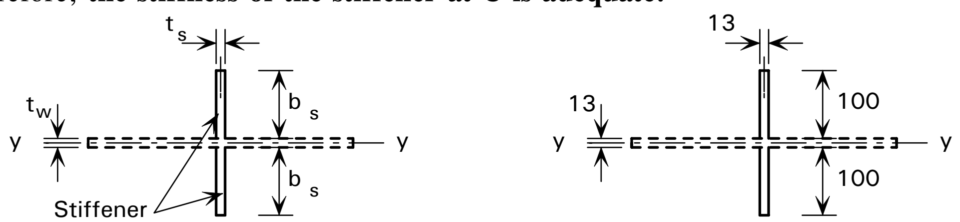


Figure 21.7 *Intermediate stiffeners at C*

Example 21 Simply supported stiffened plate girder	Sheet 10 of 13	Rev

Grade of steel = S275

$t_s \leq 16$ mm

Therefore p_y = 275 N/mm²

$$\varepsilon = \sqrt{\frac{275}{p_y}} = \sqrt{\frac{275}{275}} = 1$$

The outstand of the web stiffener should not exceed $19\varepsilon t_s$ from the face of the web. 4.5.1.2

Limiting outstand is $19\varepsilon t_s$ = 19 ×1 × 13 = 247 mm
Actual outstand is 100 mm
100 mm < 247 mm
Therefore, **the outstand is acceptable**
If the outstand of a stiffener is between $13\varepsilon t_s$ and $19\varepsilon t_s$ then its design should be based on an effective cross section with an outstand of $13\varepsilon t_s$ 4.5.1.2

Limiting length is $13\varepsilon t_s$ = 13 ×1 × 13 = 169 mm
Actual outstand is 100 mm

100 mm < 169 mm
Therefore, **the outstand is fully effective**

21.6.2 Check stiffened web at C

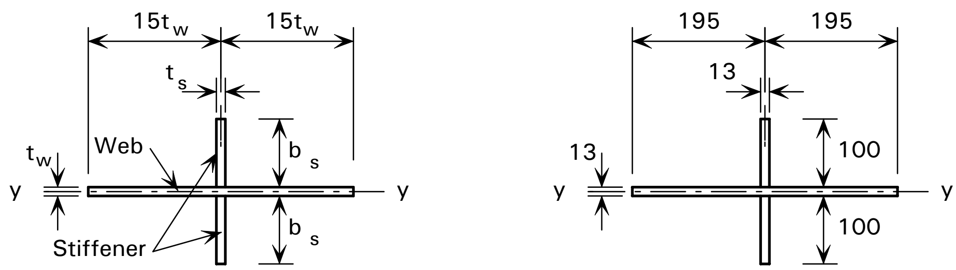


Figure 21.8 *Stiffened web at C*

Basic requirement $F_q \leq P_q$ 4.4.6.6
F_q = V - V_{cr}
F_q is the larger value obtained from panel CB and CD

Panel CB
$F_{q.CB}$ = $V_{CB} - V_{cr.CB}$
V_{CB} = 1910 kN (Figure 21.4)
$V_{cr.CB}$ = 1655 kN (see Section 21.3.3)
$F_{q.CB}$ = 1910 – 1655 = 255 kN

Panel CD
$F_{q.CD}$ = $V_{CD} - V_{cr.CD}$
V_{CD} = 1680 kN (Figure 21.4)

Calculation of $V_{cr.CD}$ for web panel CD:
d/t = 1950/13 = 150
a/d = 3000/1950 = 1.54
From Table 21, q_w = 88.9 N/mm² Table 21
V_w = dtq_w = 1950 × 13 × 88.9 = 2254 kN 4.4.5.2
A_v = td = 13 × 1950 = 25350 mm²
P_v = $0.6p_{yw} A_v$ = 0.6 × 275 × 2530 = 4183 kN 4.2.3
$0.72P_v$ = 0.72 × 4183 kN = 3012 kN

As $V_w \le 0.72\, P_v$ (2254 ≤ 3012) — 4.4.5.4

$$V_{cr.CD} = \frac{\left(\frac{V_w}{0.9}\right)^2}{P_v} = \frac{\left(\frac{2254}{0.9}\right)^2}{4183} = 1499 \text{ kN}$$

$F_{q.CD}$ = 1680 – 1499 = 181 kN

F_q = max($F_{q.CB}$, $F_{q.CD}$)

∴ F_q = 255 kN

For calculation of P_q, referring to Figure 21.8

$$I_y = \frac{13\times213^3}{12} + \frac{390\times13^3}{12} - \frac{13\times13^3}{12} = 1054 \times 10^4 \text{ mm}^4$$

$$A_s = 2\times100\times13 + 2\times195\times13 = 7670 \text{ mm}^2$$

$$r_y = \left(\frac{I}{A_s}\right)^{0.5} = \left(\frac{1054\times10^4}{7670}\right)^{0.5} = 37.1 \text{ mm}$$

$$L_E = 0.7\,L = 0.7\times1950 = 1365 \text{ mm}$$ — 4.5.5

$$\lambda = \frac{L_E}{r_y} = \frac{1365}{37.1} = 36.8$$

From Table 24, p_c = 244 N/mm² — Table 24 strut curve C

P_x = $A_s p_c$ = 7670 × 244 × 10^{-3} = 1872 kN — 4.5.5

P_q = P_x = 1872 kN

F_q = 255 kN ≤ 1872 kN

Therefore, **the buckling resistance of the stiffener is adequate.**

21.7 Stiffener at E

4.4.6

The stiffener at E should be checked as:

(1) A bearing stiffener (see clauses 4.5.1.1(a), 4.5.2)

(2) An intermediate transverse web stiffener (see clauses 4.5.1.1(d), 4.5.5, 4.4.6.2)

21.7.1 Choose a suitable stiffener size

Try stiffener 2 flats 100 mm x 13 mm grade S275 as for stiffener at C

Minimum stiffness requirement: — 4.4.6.5(a)

Since the transverse force applied is in line with web, there is no increase in minimum I_s required (calculations similar to 21.6.1 but not shown here).

21.7.2 Check bearing capacity of the stiffened web

Basic requirement: (F_x –P_{bw}) ≤ P_s

Allowing 12 mm cope for web/flange weld — 4.5.2

$A_{s.net}$ = 2 × 88 × 13 = 2288 mm²

P_s = $A_{s.net}\,p_y$ = 2288 × 275 × 10^{-3} = 629 kN

P_{bw} = 715 kN (same as in 21.4.2)

F_x = 760 kN

F_x - P_{bw} = 760 - 715 = 45 kN

(F_x –P_{bw}) = 45 kN < 629 kN

Therefore, **the bearing capacity of the stiffened web is adequate**.

21.7.3 Check buckling resistance of intermediate transverse web stiffeners subject to external forces

4.4.6.6

If $F_q \leq F_x$ then basic requirement is $\dfrac{F_x}{P_x} + \dfrac{M_s}{M_{ys}} \leq 1$

F_x = 760 kN

F_q is the larger value obtained from panels ED and EF.

Panel ED

$F_{q.ED} = V_{ED} - V_{cr.ED}$

V_{ED} = 1404 kN (see Figure 21.4)

d/t = 1950/13 = 150

a/d = 3500/1950 = 1.79

From Table 21, q_w = 86 N/mm² — Table 21

$V_{w.ED} = dtq_w = 1950 \times 13 \times 86 \times 10^{-3} = 2180$ kN

$P_v = 0.6p_{yw}A_v = 0.6 \times 275 \times 25350 \times 10^{-3} = 4183$ kN

$0.72P_v$ = 3012 kN

As $V_w \leq 0.72P_v$ (2180 kN ≤ 3012 kN)

$$V_{cr.ED} = \frac{\left(\dfrac{V_w}{0.9}\right)^2}{P_v} = \frac{\left(\dfrac{2180}{0.9}\right)^2}{4183} = 1403 \text{ kN}$$

$F_{q.ED}$ = 1404 – 1403 = 1 kN

Panel EF

$F_{q.EF} = V_{EF} - V_{cr.EF}$

V_{EF} = 1082 kN (see Figure 21.4)

d/t = 1950/13 = 150

a/d = 3500/1950 = 1.79

From Table 21, q_w = 86 N/mm² — Table 21

$V_{w.EF} = dtq_w = 1950 \times 13 \times 86 \times 10^{-3} = 2180$ kN

$P_v = 0.6p_{yw}A_v = 0.6 \times 275 \times 25350 \times 10^{-3} = 4183$ kN

$0.72P_v$ = 3012 kN

As $V_w \leq 0.72P_v$ (2180 kN ≤ 3012 kN)

$$V_{cr.EF} = \frac{\left(\dfrac{V_w}{0.9}\right)^2}{P_v} = \frac{\left(\dfrac{2180}{0.9}\right)^2}{4183} = 1403 \text{ kN}$$

$F_{q.EF}$ = 1082 – 1403 = -321 kN

(Note: As $F_{q.EF}$ is negative, there is no tension field action)

$F_q = \max(F_{qED}, F_{qEF})$

$\therefore F_q$ = 1 kN

Therefore $\boldsymbol{F_q \leq F_x}$

As $F_q \leq F_x$, the basic requirement is $\dfrac{F_x}{P_x} + \dfrac{M_s}{M_{ys}} \leq 1$

F_x = 760 kN

P_x = 1872 kN (as calculated for C)

M_s = 0 kN

F_x/P_x = 760/1872 = 0.41

0.41 < 1

Therefore, **the buckling resistance of the stiffener is adequate.**

Example 21 Simply supported stiffened plate girder	Sheet 13 of 13	Rev

21.8 Web check between stiffeners

This additional check should be carried out where load is applied between two stiffeners.

Basic requirement $f_{ed} \le p_{ed}$ 4.5.3.2

Since the UDL is constant, it is only necessary to check the largest panel.

$f_{ed} = w'/t = 92/13 = 7.1$ N/mm^2

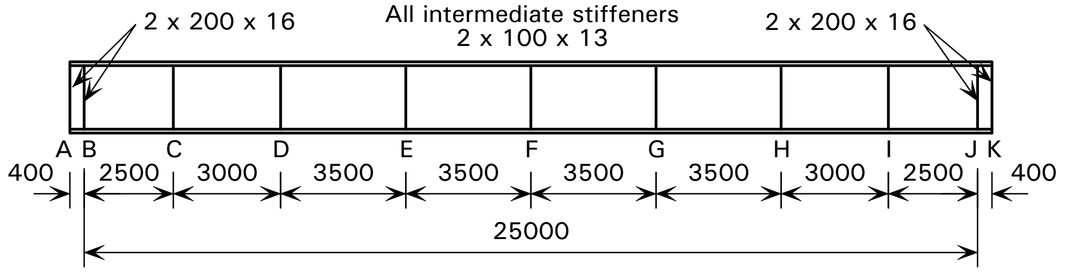
It is assumed that the compression flange is restrained against rotation relative to the web.

$$p_{ed} = \left(2.75 + \frac{2}{\left(\frac{a}{d}\right)^2}\right)\frac{E}{\left(\frac{d}{t}\right)^2} = \left(2.75 + \frac{2}{\left(\frac{3500}{1950}\right)^2}\right)\frac{205000}{\left(\frac{1950}{13}\right)^2} = 30.7 \text{ N/mm}^2$$

$f_{ed} = 7.1$ N/mm$^2 \le 30.7$ N/mm^2

Therefore, **the web is adequate for all panels.**

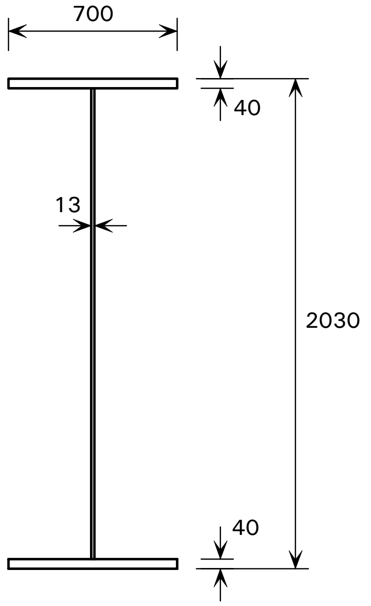
21.9 Final plate girder details



Note: (1) The size of the intermediate stiffeners could be reduced, but practical stiffener sizes have been adopted

(2) Intermediate stiffener at F could be left out.

(3) Other positions of stiffeners are possible.



All material grade S 275

Figure 21.9 *Final plate girder details*

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CALCULATION SHEET

Job No. CDS 153		Sheet 1 of 6	Rev
Job Title Example no. 22			
Subject Sway stability			
Client SCI	Made by MDH	Date Jun 2003	
	Checked by ASM	Date Oct 2003	

22 Sway stability

22.1 Introduction

The frame shown in Figure 22.1 consists of steel beams and columns arranged on a 7 m × 7 m grid. The frame has been designed on the basis of "Simple Design" according to BS 5950-1:2000[1] clause 2.1.2.2. Resistance to sway is provided by two 3.5 m braced bays, one on each 49 m side, as shown in Figure 22.2. In practice, bracing would also be required parallel to the 28 m side, but this is not considered in this example. Once constructed, the frame will be clad to form an office block, but the stiffening effect of the cladding has not been taken into account in the analysis of the frame. Determine whether the frame is "non-sway" or "sway sensitive" according to BS 5950-1:2000[1] and, if necessary, calculate the amplification factor k_{amp}.

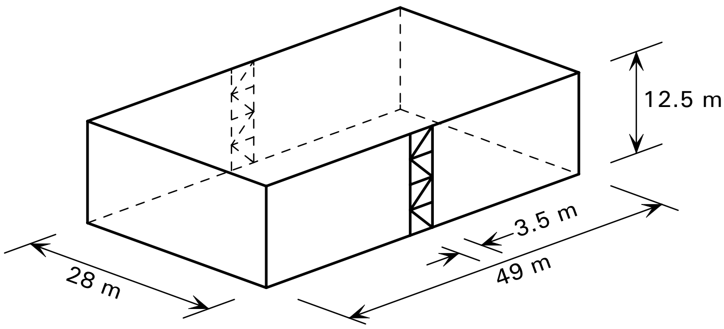

Figure 22.1

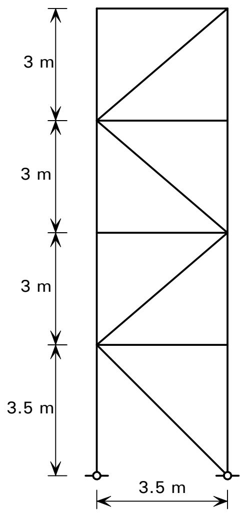

Figure 22.2

22.1.1 Unfactored roof and floor loads

Roof:
Dead load w_{dr} = 3.5 kN/m²
Imposed load w_{ir} = 1.0 kN/m²

Floor:
Dead load w_{df} = 3.5 kN/m²
Imposed load w_{if} = 6.0 kN/m²

Example 22 Sway stability	Sheet 2 of 6	Rev

22.1.2 Factored roof and floor loads

Consider the following three load combinations:

(1) 1.4 dead + 1.6 imposed

(2) 1.0 dead + 1.4 wind (dead load resisting overturning due to wind)

(3) 1.2 dead + 1.2 imposed + 1.2 wind

Gravity loads for load combination 1

Roof: $w_r' = (3.5 \times 1.4) + (1.0 \times 1.6) = 6.5$ kN/m^2
Floor: $w_f' = (3.5 \times 1.4) + (6.0 \times 1.6) = 14.5$ kN/m^2

Gravity loads for load combination 2

Roof: $w_r' = 3.5 \times 1.0 = 3.5$ kN/m^2
Floor: $w_f' = 3.5 \times 1.0 = 3.5$ kN/m^2

Gravity loads for load combination 3

Roof: $w_r' = (3.5 \times 1.2) + (1.0 \times 1.2) = 5.4$ kN/m^2
Floor: $w_f' = (3.5 \times 1.2) + (6.0 \times 1.2) = 11.4$ kN/m^2

22.1.3 Member sizes

Roof beam	305 × 127 × 37 UB in grade S275
Floor beam	406 × 178 × 60 UB in grade S275
Ground to 2nd floor columns	203 × 203 × 60 UC in grade S275
2nd floor to roof columns	203 × 203 × 46 UC in grade S275
Bracing	168.3 × 6.3 CHS in grade S275

22.2 Sway Stability

The sway stability of the structure is assessed by performing an elastic analysis on one of the braced bays, under the action of the notional horizontal forces (NHF), according to the rules in clause 2.4.2 of BS 5950-1:2000[1]. The notional horizontal forces are applied as horizontal point loads at every roof and floor level and are taken as 0.5% of the total factored dead + imposed loads for that level. In this example, since the stability is provided by two braced bays, the notional horizontal forces applied to one bracing system should be taken as half the value calculated for the whole floor or roof. 2.4.2.4

The greatest notional horizontal forces occur in load combination 1 and this case should generally be used when assessing sway stability. However, advantage may be taken of the lower notional horizontal forces in load combinations 2 and 3, if desired. All three combinations are considered below.

22.2.1 Load combination 1 (Dead + Imposed)

Roof level NHF $= 0.005 \times 0.5 \times 28 \times 49 \times 6.5 = 22.3$ kN 2.4.2.4
Floor level NHF $= 0.005 \times 0.5 \times 28 \times 49 \times 14.5 = 49.7$ kN

The result of an elastic analysis on one braced bay (bare frame only) under the action of the notional horizontal forces is shown in Figure 22.3.

Example 22 Sway stability	Sheet 3 of 6	Rev

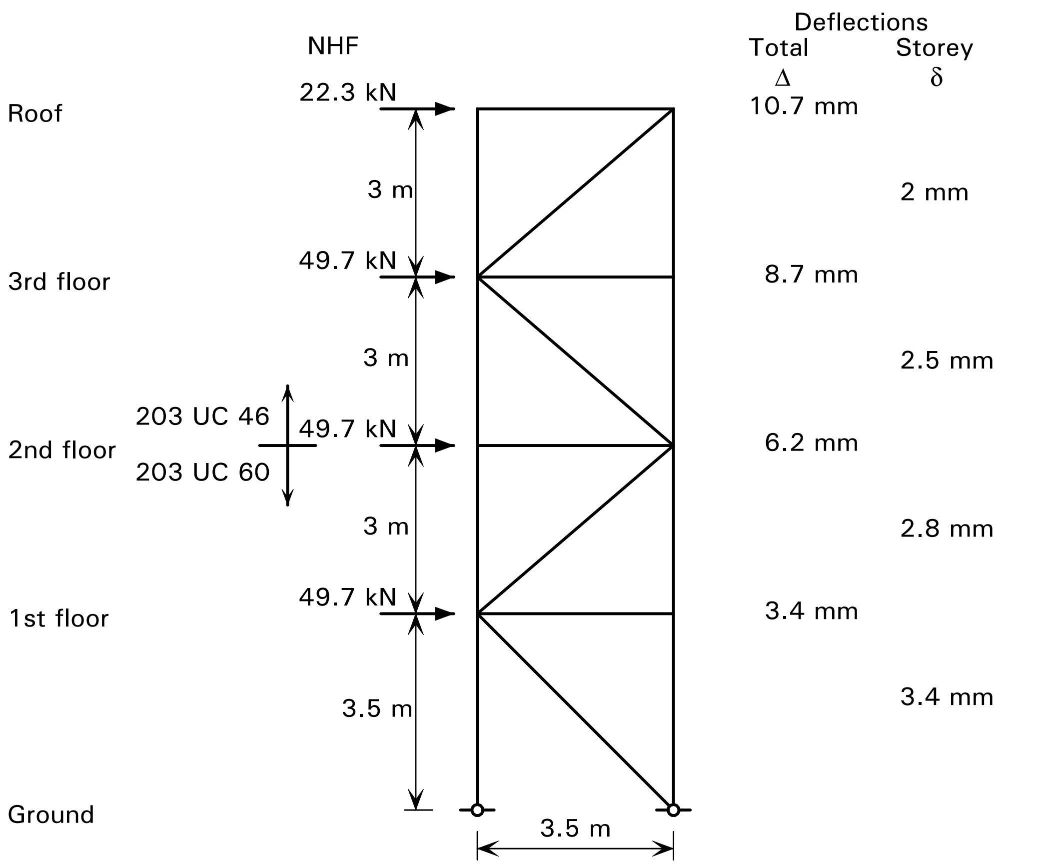

Figure 22.3 *Deflections due to NHF for load combination 1*

For multi-storey frames, an approximate value of the elastic critical buckling factor λ_{cr} is obtained from:

$$\lambda_{cr} = \frac{h}{200\,\delta}$$ 2.4.2.6

where δ is the inter-storey lateral drift (relative deflection between two adjacent storeys) and h is the storey height.

In assessing whether a frame is "non-sway" or "sway sensitive", the smallest value of λ_{cr} should be used. By inspection, it can be seen that the smallest λ_{cr} will either be between ground and first floors or between the first and second floors.

Ground – 1st floor: $\lambda_{cr} = \dfrac{3500}{200 \times 3.4} = 5.15$ 2.4.2.6

1st floor – 2nd floor: $\lambda_{cr} = \dfrac{3000}{200 \times 2.8} = 5.36$

Therefore, $\lambda_{cr} = 5.15$.

Note: The critical value of λ_{cr} usually occurs between the ground and first floors, due to the fact that there is no deflection at ground level (high lateral drift between ground and 1st floor).

Since $\lambda_{cr} < 10$, the frame is classed as "sway sensitive" for load combination 1. 2.4.2.6

Example 22 Sway stability	Sheet 4 of 6	Rev

For clad frames in which the stiffening effect of the cladding has been ignored, the amplifier k_{amp} is given by:

$$k_{amp} = \frac{\lambda_{cr}}{1.15\lambda_{cr} - 1.5} \quad \text{but } k_{amp} \geq 1.0$$ 2.4.2.7(1)

$$k_{amp} = \frac{5.15}{(1.15 \times 5.15) - 1.5} = 1.16$$

Therefore, **the forces in the bracing system must be increased by 16% for load combination 1.**

22.2.2 Load combination 2 (Dead + Wind)

Roof level NHF $= 0.005 \times 0.5 \times 28 \times 49 \times 3.5 = 12.0$ kN 2.4.2.4

Floor level NHF $= 0.005 \times 0.5 \times 28 \times 49 \times 3.5 = 12.0$ kN

The result of an elastic analysis on one braced bay (bare frame only) under the action of the notional horizontal forces is shown in Figure 22.4.

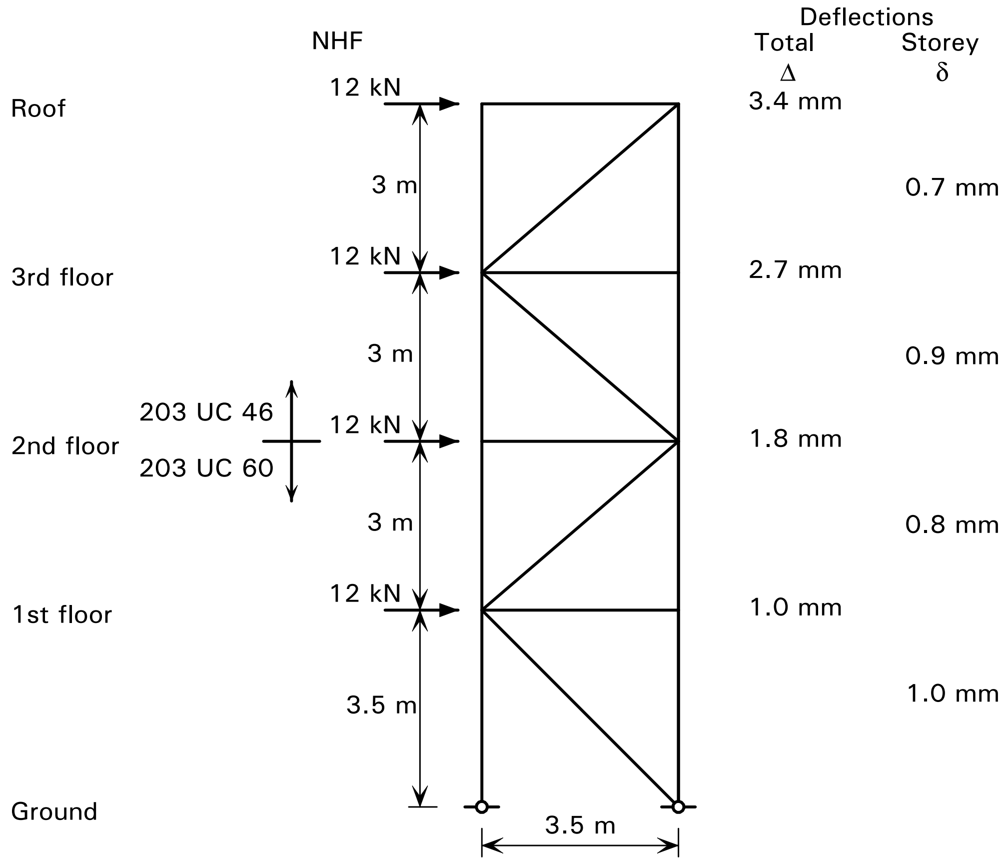

Figure 22.4 *Deflections due to NHF for load combination 2*

Ground – 1st floor: $\lambda_{cr} = \frac{3500}{200 \times 1.0} = 17.5$

2nd floor – 3rd floor: $\lambda_{cr} = \frac{3000}{200 \times 0.9} = 16.7$

Therefore, $\lambda_{cr} = 16.7$

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Since λ_{cr} > 10, the frame is classed as "non-sway" and there is no need to amplify the forces in the bracing system for load combination 2. 2.4.2.6

Note: A frame with λ_{cr} > 10 is only classed as "non-sway" if it is a clad frame that has been analysed as a bare frame in determining λ_{cr}. Bare frames or clad frames where the stiffness of the cladding is allowed for in determining λ_{cr} are always classified as "sway sensitive", irrespective of the value of λ_{cr}.

22.2.3 Load combination 3 (Dead + Wind + Imposed)

2.4.2.4

Roof level NHF $= 0.005 \times 0.5 \times 28 \times 49 \times 5.4 = 18.5$kN

Floor level NHF $= 0.005 \times 0.5 \times 28 \times 49 \times 11.4 = 39.1$kN

The result of an elastic analysis on one braced bay (bare frame only) under the action of the notional horizontal forces is shown in Figure 22.5.

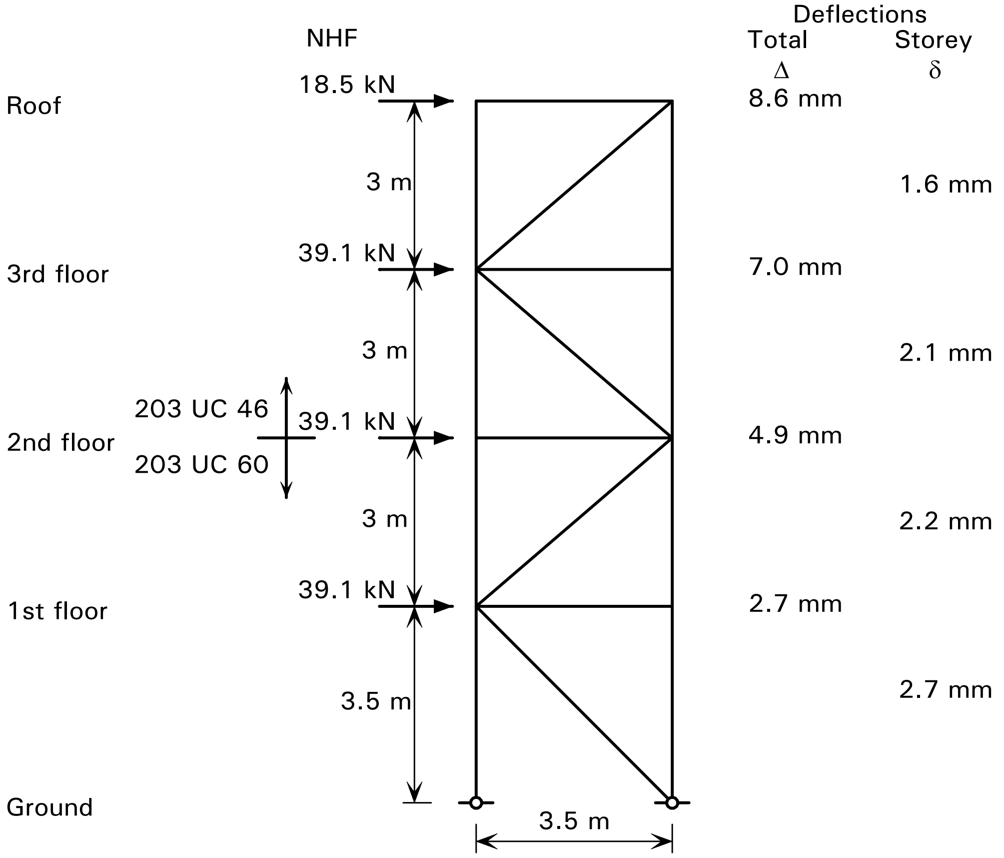

Figure 22.5 *Deflections due to NHF for load combination 3*

Ground – 1st floor: $\lambda_{cr} = \dfrac{3500}{200 \times 2.7} = 6.48$ 2.4.2.6

1st floor – 2nd floor: $\lambda_{cr} = \dfrac{3000}{200 \times 2.2} = 6.82$

Therefore, $\lambda_{cr} = 6.48$

Since λ_{cr} < 10, the frame is classed as "sway sensitive" for load combination 3.

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For clad frames in which the stiffening effect of the cladding has been ignored, the amplifier k_{amp} is given by:

$$k_{amp} = \frac{\lambda_{cr}}{1.15\lambda_{cr} - 1.5} = \frac{6.48}{(1.15 \times 6.48) - 1.5} = 1.09$$ 2.4.2.7 (1)

Therefore, the forces in the bracing system must be increased by 9% for load combination 3.

The Steel Construction Institute Silwood Park, Ascot, Berks SL5 7QN Telephone: (01344) 623345 Fax: (01344) 622944 **CALCULATION SHEET**	Job No. CDS 153	Sheet 1 of 12	Rev
	Job Title Example no. 23		
	Subject Tying and the avoidance of disproportionate collapse		
	Client SCI	Made by MDH	Date Jun 2003
		Checked by ASM	Date Oct 2003

23 Tying and the avoidance of disproportionate collapse

23.1 Introduction

The ten-storey building shown in Figure 23.1 has been designed on the basis of "Simple Design" in accordance with the recommendations of BS 5950-1:2000[1]. All storeys are 4.0 m high, apart from the ground to first floor, which has a height of 5.0 m. The columns are laid out on a 6 m × 9 m grid with the primary beams spanning 6 m and the secondary beams spanning 9 m as shown in Figure 23.2. The spacing of the secondary beams is 3.0 m. A composite flooring system is used with steel decking spanning between the secondary beams. All the secondary and primary beams are assumed to act compositely with the floor slab. The steel frame is of simple construction, with two braced bays on each of the four sides providing lateral stability.

Check that the building meets the requirements of BS 5950-1:2000[1] in terms of structural integrity and the avoidance of disproportionate collapse.

2.4.5

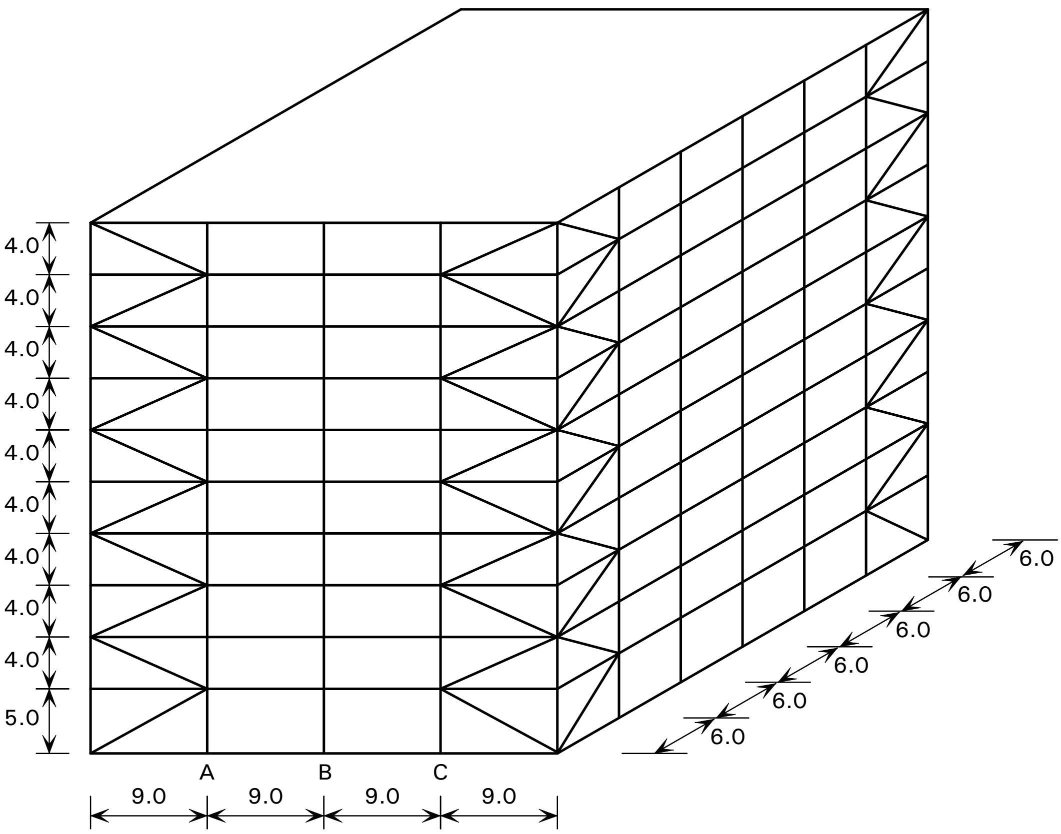

Figure 23.1

In the first instance, check for integrity is achieved by ensuring that the five conditions listed in sub-clause 2.4.5.3 of BS 5950-1:2000[1] are satisfied. Where this is not possible, the designer must check that the removal of any individual member does not lead to disproportionate collapse as defined in BS 5950-1:2000[1]. Finally, if the removal of a member would cause disproportionate collapse, this member must be designed as a key element. All three stages of this process are demonstrated in this example.

In practice, these checks must be carried out on all members to ensure adequate robustness throughout the structure. However, in this example, the checks are only performed on a typical secondary beam, an edge column and an internal column. These columns are denoted B and E respectively in Figure 23.2.

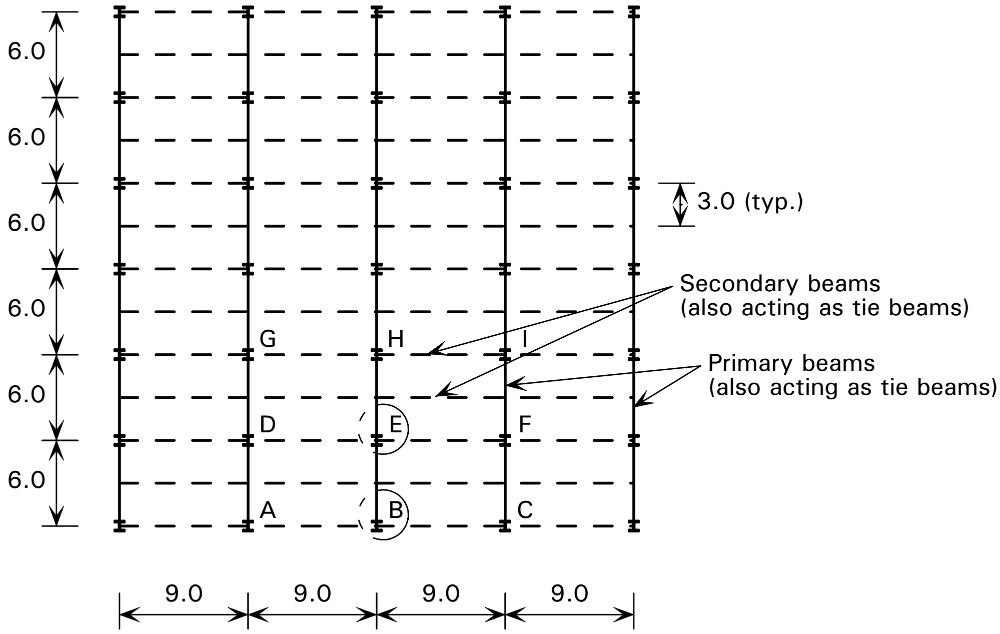

Figure 23.2

The composite floor system comprises steel decking spanning between the secondary beams, as shown in Figure 23.3, with a 125 mm thick slab in grade C30 concrete.

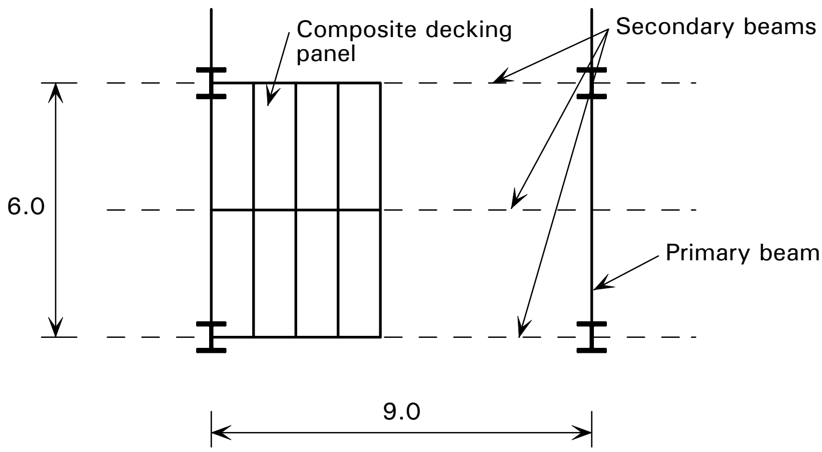

Figure 23.3

Example 23 Tying and the avoidance of disproportionate collapse	Sheet 3 of 12	Rev

23.1.1 Unfactored roof and floor loads

Dead load (assume the same for roof and floor)

S/w concrete = 2.67 kN/m^2
S/w decking = 0.17 kN/m^2
S/w beams = 0.15 kN/m^2
Total s/w = 2.99 kN/m^2

Allow 0.5 kN/m^2 for ceilings and services.

Total unfactored dead load = 3.49 kN/m^2

Imposed load

Roof: 1.0 kN/m^2
Floor: 5.0 kN/m^2 + 1.0 kN/m^2 (partitions) = 6.0 kN/m^2

23.1.2 Unfactored cladding loads

The external beams carry a brick and block cavity wall plastered on one side. From BS 648:1964[8], the weight of the wall is 3.76 kN/m^2. Since the storey height is 4.0 m, the distributed load on each external beam is

BS 648

w_{clad} = 4.0 × 3.76 = 15.04 kN/m

23.1.3 Factored roof loads

Edge column B

Edge columns support an area of 27 m^2

$W'_{roof,B}$ = ((3.49 × 1.4) + (1.0 × 1.6)) × 27 = 175 kN

Internal column E

Internal columns support an area of 54 m^2

$W'_{roof,E}$ = ((3.49 × 1.4) + (1.0 × 1.6)) × 54 = 350 kN

23.1.4 Factored floor loads on secondary beams

Edge beams

w' = (15.04 × 1.4) + (1.5 × 3.49 × 1.4) + (1.5 × 6.0 × 1.6) = 42.8 kN/m
Total load per beam = 42.8 × 9.0 = 385.2 kN

Internal beams

w' = (3.0 × 3.49 × 1.4) + (3.0 × 6.0 × 1.6) = 43.5 kN/m
Total load per beam = 43.5 × 9.0 = 391.5 kN

23.1.5 Factored floor loads on primary beams

It is assumed that the entire slab loading is carried by the secondary beams and then transferred to the primary beams as point loads. Therefore, the only loads applied to the primary beams are the internal secondary beam reactions. Each internal primary beam supports two secondary beams.

Total load per beam = 2 × 0.5 × 391.5 = 391.5 kN

23.1.6 Factored floor loads on columns

Edge column B

Column B supports 2 edge beams and 1 primary beam and carries half the load from each beam.

$W'_{\text{floor,B}} = (2 \times 0.5 \times 385.2) + (0.5 \times 391.5) = 581$ kN

Internal column E

Column E supports 2 internal secondary beams and 2 internal primary beams and carries half the load from each beam.

$W'_{\text{floor,E}} = (2 \times 0.5 \times 391.5) + (2 \times 0.5 \times 391.5) = 783$ kN

23.2 Member sizes

The composite beams were designed using the BDES* software and the column sizes were estimated using the member capacity tables in Volume 1[2]. In sizing the beams, the final composite condition and the construction stage non-composite condition were both checked. Since the internal and external beams experience similar loading, only the internal beams were considered. For simplicity, the columns were sized for compression only. In practice, they would have to be designed as columns in simple construction, following the procedure outlined in Example 14.

*Available from www.corusconstruction.com/page_679.htm

23.2.1 Beam sizes

Secondary beams

406 × 140 × 46 UB in grade S355.

Primary beams

457 × 152 × 52 UB in grade S355.

23.2.2 Column sizes

The factored loading, effective lengths and selected column sizes for columns B and E are given in Tables 23.1 and 23.2 respectively.

Table 23.1 Edge column B

Column location	L_{eff} (m)	Factored load ex. column s/w (kN)	Selected section (all S355)	Factored load inc. column s/w (kN)	Resistance (kN)
Roof-9	4.0	175	305×305×97UC	175	3310
9-8	4.0	756	305×305×97UC	761	3310
8-7	4.0	1337	305×305×97UC	1348	3310
7-6	4.0	1918	305×305×97UC	1934	3310
6-5	4.0	2499	305×305×97UC	2520	3310
5-4	4.0	3080	305×305×97UC	3107	3310
4-3	4.0	3661	305×305×137UC	3693	4620
3-2	4.0	4242	305×305×137UC	4282	4620
2-1	4.0	4823	305×305×198UC	4870	6780
1-0	5.0	5404	305×305×198UC	5462	5920

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Table 23.2 Internal column E

Column location	L_{eff} (m)	Factored load ex. column s/w (kN)	Selected section (all S355)	Factored load inc. column s/w (kN)	Resistance (kN)
Roof-9	4.0	350	305×305×97UC	350	3310
9-8	4.0	1133	305×305×97UC	1138	3310
8-7	4.0	1916	305×305×97UC	1927	3310
7-6	4.0	2699	305×305×97UC	2716	3310
6-5	4.0	3482	305×305×137UC	3504	4620
5-4	4.0	4265	305×305×137UC	4295	4620
4-3	4.0	5048	305×305×198UC	5085	6780
3-2	4.0	5831	305×305×198UC	5879	6780
2-1	4.0	6614	305×305×283UC	6674	9200
1-0	5.0	7397	305×305×283UC	7473	8030

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It is assumed that the columns are spliced every two storeys and that lateral restraint is provided at every floor. It is further assumed that the columns may be treated as pin-ended between the floor levels.

23.3 Disproportionate collapse checks using fin plate beam-to-column connections

Regulations that stipulate that certain buildings should be designed to avoid disproportionate collapse may be assumed to be satisfied if the five conditions listed in 2.4.5.3 of BS 5950-1:2000[1] are met. These five conditions are considered in Sections 23.3.1 to 23.3.5 below. It is assumed that fin plates are used for all beam-to-column connections.

23.3.1 General tying

2.4.5.3 a)

Horizontal ties should be arranged in continuous lines throughout each floor and roof level in two approximately perpendicular directions. All members acting as ties and their end connections should be designed to resist a tensile force equal to the end reaction of the member under factored loads or 75 kN, whichever is greater.

Typical secondary beam (406 × 140 × 46 UB in grade S355)

The connection should be designed to resist the beam-to-column reaction in shear and then checked to ensure that it has an adequate tying capacity. The values of shear capacity and tying capacity used in this example have been obtained from *Joints in steel construction: Simple connections*[6] and are based on the steel connection alone. No allowance has been made for the capacity of the reinforcement in the concrete to carry some of the load.

Try 290 x 150 x 10 mm fin plate in S275 with two lines of 4 bolts.

Basic requirement 1: Reaction ≤ Shear capacity

Reaction under factored loads = 196 kN

Shear capacity = 289 kN

196 kN < 289 kN

Therefore **the shear capacity is adequate.**

Table H.30 P212[6]

Basic requirement 2: Tying force ≤ Tying capacity

Required tying force = 196 kN

Tying capacity = 488 kN

196 kN < 488 kN

Therefore **the tying capacity is adequate.**

Table H.30 P212 [6]

Typical primary beam (457 x 152 x 52 UB in grade S355)

Try 290 x 100 x 10 mm fin plate in S275 with a single line of 4 bolts.

Basic requirement 1: Reaction ≤ Shear capacity

Reaction under factored loads = 196 kN

Shear capacity = 212 kN

196 kN <212 kN

Therefore **the shear capacity is adequate.**

Table H.29 P212 [6]

Basic requirement 2: Tying force ≤ Tying capacity

Required tying force = 196 kN

Tying capacity = 334 kN

196 kN < 334 kN

Therefore **the tying capacity is adequate.**

Table H.29 P212 [6]

23.3.2 Tying of edge columns

Horizontal ties should be provided to hold the vertical perimeter columns in position. These ties should be capable of resisting a tying force, acting perpendicular to the edge, equal to the greater of 1% of the maximum factored vertical load in the column adjacent to that level or the load specified in the general tying requirement.

2.4.5.3 b)

Consider the lowest level, where the load in the column is greatest.

From Table 23.1, the load in the column = 5462 kN

1% of 5462 kN = 54.62 kN

The tying force specified in 23.3.1 = 196 kN.

Therefore, **the general tying requirement is critical in this case.**

23.3.3 Continuity of columns

The column splices should be capable of resisting a tensile force equal to the largest factored vertical reaction applied to the column at a single floor level located between the column splice under consideration and the next column splice down.

2.4.5.3 c)

Basic requirement: Applied vertical floor load ≤ Column splice capacity

For the purpose of this example, consider column E as a typical internal column. The factored vertical load applied to the column from each floor is

$W'_{\text{floor,E}}$ = 783 kN.

Seet 4 (23.1.6)

Consider the weakest splice in that column (that between 305 × 305 × 97UC and 305 × 305 × 97UC). From Table H.32 of *Joints in steel construction: Simple connections*[6], the capacity of one external flange cover plate is 794 kN.

Table H.32 P212 [6]

Splice capacity = 2 × 794 kN = 1588 kN

783 kN < 1588 kN **Therefore the column splice capacity is adequate.**

23.3.4 Resistance to horizontal forces

Condition d) of clause 2.4.5.3 states that there must be more than one system of bracing stabilizing the structure in two approximately orthogonal directions. In this Example, this is satisfied by the braced bays shown in Figure 23.1 — 2.4.5.3 d)

23.3.5 Heavy floor units

Heavy precast floor units are not used in this Example, so condition e) of clause 2.4.5.3 does not apply. If heavy precast floor units are used, the designer must ensure that they are sufficiently secure against dislodgement. — 2.4.5.3 e)

23.3.6 Conclusion

Having satisfied the five conditions in clause 2.4.5.3 of BS 5950-1:2000[1], it may be assumed that this building meets the requirements of the regulations for the avoidance of disproportionate collapse.

23.4 Disproportionate collapse checks using flexible end plate beam-to-column connections

As an alternative to fin plate connections, this section considers the use of flexible end plates.

23.4.1 General tying

Typical secondary beam (406 x 140 x 46 UB in grade S355) — 2.4.5.3 a)

Try 290 x 150 x 8 mm flexible end plate in S275 with 4 rows of bolts.

Basic requirement 1: Reaction ≤ Shear capacity — Table H.21 P212 [6]

Reaction under factored loads = 196 kN
Shear capacity = 378 kN
196 kN < 378 kN
Therefore **the shear capacity is adequate.**

Basic requirement 2: Tying force ≤ Tying capacity — Table H.21 P212 [6]

Required tying force = 196 kN
Tying capacity = 226 kN
196 kN < 226 kN
Therefore **the tying capacity is adequate.**

Now consider the situation in which the designer is unable to use such a deep secondary beam and opts instead for the slightly heavier 305 x 165 x 54 UB in S355.

Try 290 x 150 x 8 mm flexible end plate in S275 with 3 rows of bolts.

Basic requirement 1: Reaction ≤ Shear capacity — Table H.21 P212 [6]

Reaction under factored loads = 196 kN
Shear capacity = 333 kN
196 kN < 333 kN
Therefore **the shear capacity is adequate.**

Basic requirement 2: Tying force ≤ Tying capacity — Table H.21 P212 [6]

Required tying force = 196 kN
Tying capacity = 175 kN
196 kN > 175 kN
Therefore **the tying capacity is NOT adequate** and it is necessary to check for disproportionate collapse.

Example 23 Tying and the avoidance of disproportionate collapse	Sheet 8 of 12	Rev

23.4.2 Check for disproportionate collapse

If any of the first three conditions listed in clause 2.4.5.3 of BS 5950-1:2000 are not satisfied, the building should be checked to ensure that the removal of any one column would not lead to disproportionate collapse. Collapse is said to be disproportionate if at any given level it exceeds 15% of the floor or roof area or 70 m^2. For the purpose of this example, this check has been restricted to column E. In practice, each column should be checked in turn. 2.4.5.3

The checks performed in Section 23.4.1 have already established that the tying capacity of the flexible end plate connections is inadequate, so the current check becomes one of measuring the area supported by the column. In this case, the removal of column E would lead to the collapse of a section of floor measuring 12 m × 18 m, i.e. 216 m^2. Therefore, **there is a risk of disproportionate collapse** and the member should be designed as a key element using the accident loading specified in BS 6399-1[7], i.e. 34 kN/m^2.

23.4.3 Key element design

The area to which the accidental loading is applied is dependent on the type of cladding or floor decking and, in particular, its integrity under blast loading. In this example, it is assumed that there is partitioning running between columns D, E and F, but none in the perpendicular direction. As the partitioning is not load-bearing, it is reasonable to assume that it is mostly blown out by the blast, leaving only a small section as shown in Figure 23.4. In this case, the breadth of partitioning remaining after the blast is assumed to be B+200 mm. 2.4.5.4

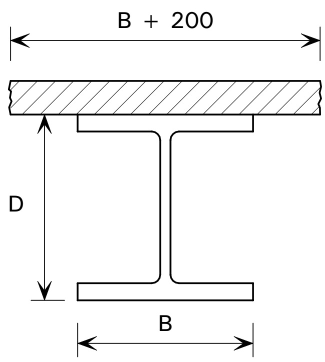

Figure 23.4

In the design of key elements, the accidental loading should be applied in all directions, but only in one direction at a time. This means checking column E in bending about both the major and minor axes. The ordinary dead and imposed loads must also be taken into account (there is no wind loading on column E) and should be applied simultaneously with the accidental loading. However, the imposed load can be reduced to one third of its normal value for this check, with a γ_f factor of 1.05. The same γ_f should also be applied to the dead load, but the accidental load should not be factored. 2.4.5.3

All of the calculations below relate to the column length between ground and first floor levels. In practice, all levels should be checked.

Section properties

The size of the internal column between ground and first floor levels is **305 x 305 x 283 UC, grade S355**.

From section property tables:

Depth	D	= 365.3 mm	Vol 1 Page B-8
Width	B	= 322.2 mm	
Web thickness	t	= 26.8 mm	
Flange thickness	T	= 44.1 mm	
Depth between fillets	d	= 246.7 mm	
Area of cross-section	A_g	= 360 cm^2	Vol 1 Page B-9
Plastic modulus	S_x	= 5110 cm^3	
Plastic modulus	S_y	= 2340 cm^3	
Elastic modulus	Z_x	= 4320 cm^3	
Elastic modulus	Z_y	= 1530 cm^3	
Radius of gyration	r_x	= 14.8 cm	
Radius of gyration	r_y	= 8.27 cm	

Grade of steel = S355 — 3.1.1 Table 9

40 mm $< T <$ 63 mm

Therefore p_y = 335 N/mm^2

Axial load

For the purpose of key element design, the factored axial loads applied to the column by the roof and each floor level are as follows:

Roof

W' = 1.05(3.49 + (1.0/3)) × 54 = 216.8 kN

Floor

W' = 1.05(3.49 + (6.0/3)) × 54 = 311.3 kN

Column self-weight

Unfactored column s/w = ((4×4.0×97)+(2×4.0×137)+(2×4.0×198)+(4.0×283)) × 9.81/1000 = 52.6 kN

Factored column s/w = 1.05 × 52.6 = 55.2 kN

Total factored axial load including self-weight

F_c = 216.8 +(9 × 311.3) +55.2 = 3074 kN

Section classification

According to Volume 1[2], the compact F/P_z limit for a 305 × 305 × 283UC in grade S355 is 1.0. Therefore, **the section is at least compact.** — Vol 1 PageD-126

Major axis bending

Loading

The accidental loading about the major axis is applied to the section of partitioning shown in Figure 23.4.

B = 322.2 mm. Therefore, the total loaded width = 522.2 mm

Example 23 Tying and the avoidance of disproportionate collapse	Sheet 10 of 12	Rev

Accidental loading $= 0.5222 \times 34 = 17.8$ kN/m.

The accidental load is applied uniformly along the column between ground and first floor levels, with the section of column between floors 1 and 2 unloaded. Although the column is continuous across the support at floor level 1, it is a safe approximation to take the maximum moment as $wL^2/8$, where $L = 5.0$ m. The actual moment will not be greater than this value.

$M_x = 17.8 \times 5^2 /8 = 55.6$ kNm.

Bending

4.2.5 Vol 1 Page D-126

Basic requirement: $M_x \leq M_{cx}$
From Volume 1[2] 1, $M_{cx} = 1710$ kNm
From above, $M_x = 55.6$ kNm
55.6 kNm < 1710 kNm
Therefore, the moment capacity is adequate.

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Basic requirement: $M_x \leq M_b / m_{LT}$
From Volume 1[2], for $L_E = 5.0$ m, $M_b = 1640$ kNm

m_{LT} is obtained from Table 18 of BS 5950-1:2000 [1] according to the shape of the bending moment diagram.

Table 18

$m_{LT} = 0.925$
$M_b / m_{LT} = 1640/0.925 = 1773$ kNm
From above, $M_x = 55.6$ kNm
55.6 kNm < 1773 kNm
Therefore**, the buckling resistance moment is adequate.**

Interaction checks – section capacity

4.8.3.2. a)

Basic requirement: $\dfrac{F_c}{A_g p_y} + \dfrac{M_x}{M_{cx}} + \dfrac{M_y}{M_{cy}} \leq 1$

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From Volume 1[2], $A_g p_y = 12100$ kN
From above, $F_c = 3074$ kN
There is no minor axis loading in this case so the third term in the equation can be ignored.

$$\frac{3074}{12100} + \frac{55.6}{1710} + 0 = 0.254 + 0.033 = 0.29 < 1$$

Interaction checks – member buckling

4.8.3.3.1

Basic requirement: $\dfrac{F_c}{P_c} + \dfrac{m_x M_x}{p_y Z_x} + \dfrac{m_y M_y}{p_y Z_y} \leq 1$

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For $L_E = 5.0$ m, $P_c = 8030$ kN

4.8.3.3.4 Table 26

m_x is obtained from Table 26 of BS 5950-1:2000[1].
$m_x = 0.95$

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$p_y Z_x = 1450$ kNm

$$\frac{3074}{8030} + \frac{0.95 \times 55.6}{1450} + 0.383 + 0.036 = 0.42 < 1$$

Example 23 Tying and the avoidance of disproportionate collapse	Sheet 11 of 12	Rev

Basic requirement: $\frac{F_c}{P_{cy}} + \frac{m_{LT} M_{LT}}{M_b} + \frac{m_y M_y}{p_y Z_y} \leq 1$	4.8.3.3.1
M_{LT} is the maximum major axis moment in the segment = 55.6 kNm. For L_E = 5.0 m, P_{cy} = 8030 kN	Vol 1 Page D-127
M_b = 1640 kNm m_{LT} = 0.925	Table 18
$\frac{3074}{8030} + \frac{0.925 \times 55.6}{1640} + 0 = 0.383 + 0.031 = 0.41 < 1$	
Therefore, **the column is adequate when subjected to the accidental load causing bending about the major axis.**	
Minor axis bending	
Loading	
The accidental loading about the minor axis is applied to the column and an assumed thickness of partitioning, say 50 mm.	
D = 365.3 mm. Therefore, the total loaded width = 415.3 mm, say 415 mm.	
Accidental loading = 0.415 × 34 = 14.1 kN/m.	
Once again, assume the maximum moment is given by $wL^2/8$. M_y = 14.1 × 5^2 /8 = 44.1 kNm.	
Bending	
Basic requirement: $\boldsymbol{M_y \leq M_{cy}}$ From Volume 1[2], M_{cy} = 615 kNm	4.2.5 Vol 1 Page D-126
From above, M_y = 44.1 kNm 44.1 kNm < 615 kNm	
Therefore, **the moment capacity is adequate.**	
Interaction checks – section capacity	
Basic requirement: $\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$	4.8.3.2. a)
As before, $A_g p_y$ = 12100 kN and F_c = 3074 kN	
There is no major axis loading in this case so the second term in the equation can be ignored.	
$\frac{3074}{12100} + 0 + \frac{44.1}{615} = 0.254 + 0.072 = 0.33 < 1$	

Interaction checks – member buckling

Basic requirement: $\dfrac{F_c}{P_c} + \dfrac{m_x M_x}{p_y Z_x} + \dfrac{m_y M_y}{p_y Z_y} \leq 1$ — 4.8.3.3.1

From above, the column resistance $P_c = 8030$ kN

The shape of the bending moment diagram is identical to that for major axis bending (even though the values are different). Therefore, $m_y = 0.95$.

$p_y Z_y = 512$ kNm — Vol 1 Page D-127

$$\frac{3074}{8030} + 0 + \frac{0.95 \times 44.1}{512} = 0.383 + 0.082 = 0.47 < 1$$

Basic requirement: $\dfrac{F_c}{P_{cy}} + \dfrac{m_{LT} M_{LT}}{M_b} + \dfrac{m_y M_y}{p_y Z_y} \leq 1$ — 4.8.3.3.1

$$\frac{3074}{8030} + 0 + \frac{0.95 \times 44.1}{512} = 0.383 + 0.082 = 0.47 < 1$$

Therefore, **the column is also adequate when subjected to the accidental load causing bending about the minor axis.**

Note: The calculations given above demonstrate the procedure for designing a key element. However, in the vast majority of circumstances, the recommended approach is to satisfy the tying requirements in clause 2.4.5.3 of BS 5950-1:2000. The key element route should only be followed as a last resort.

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