STRUCTURAL CALCULATIONS

Proposed Redevelopment
Westwick Lock House, Boroughbridge

REF No. 1103_102

November 2022

Design Statement

General Notes

All calculations within this document are designed in accordance with all necessary regulations and codes of practice. All materials specified within are to be used strictly in accordance with manufacturer's recommendations and current codes of practice.

Codes Used
- NHBC
- BS 648: 1964 - Weights of Building Materials
- BS 6399: Pt 1: 1996 - Loadings for Buildings
- BS 6399: Pt 3: 1988 - Imposed Roof Loadings
- BS 5950: Pt 1: 2000 - Structural Steel
- BS 5628: Pt 1: 2005 - Masonry
- BS 5268: Pt 2: 2002 - Timber
- BS 8110: Pt 1: 1997 - Concrete

All works are to be carried out in accordance with all relevant CDM regulations, the health and safety guidelines and building regulations

Prior to any demolition of the existing structure an inspection is to be carried out to confirm any load bearing walls.

Nominal door and window openings to have standard steel or precast concrete lintels to suit.

All steelwork to be in accordance with CE marking standards, minimum CC2 and EXC2 classes

All steelwork to be minimum grade S355 unless specified.
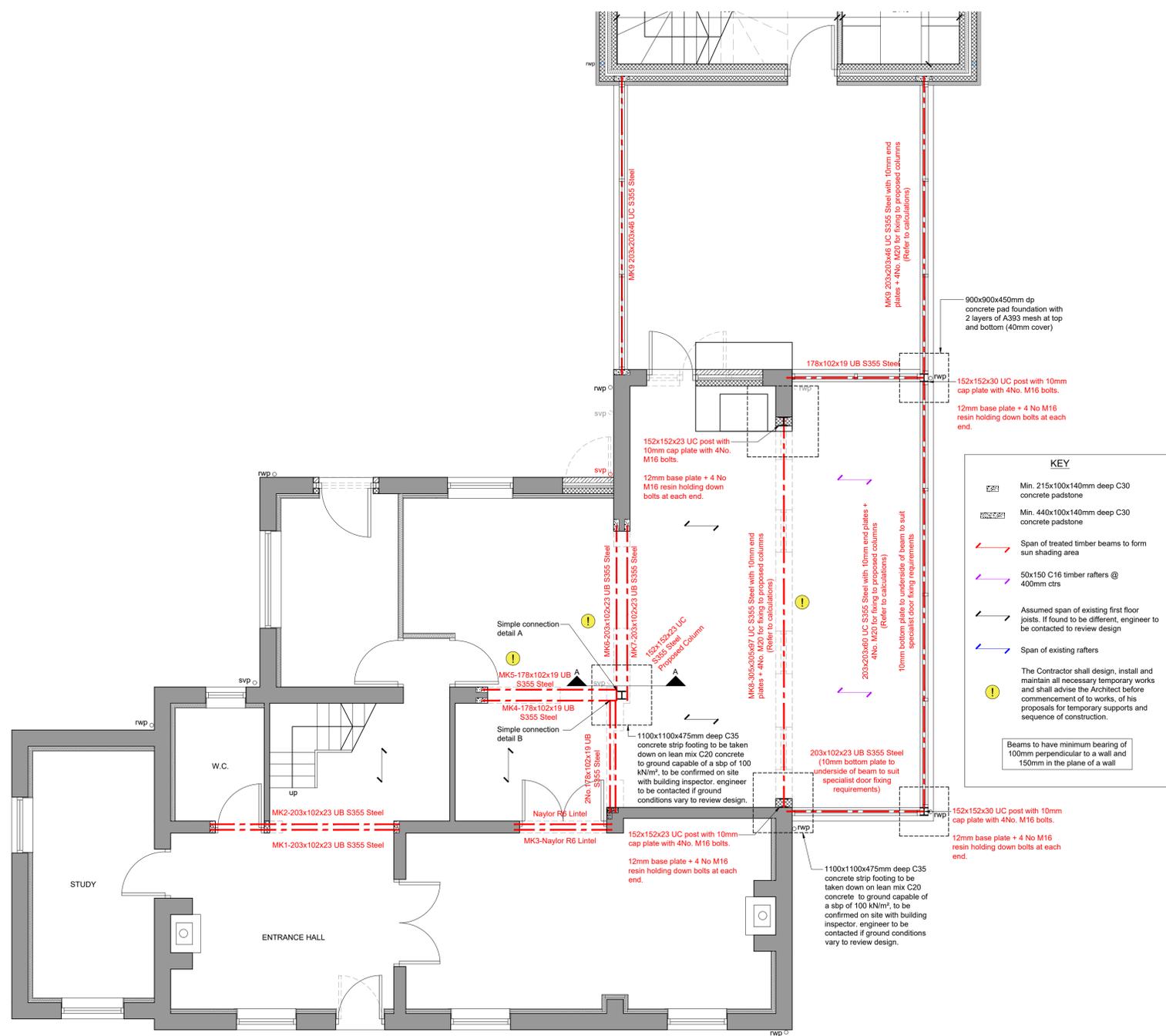
All steelwork connections to be agreed with fabricator.

Steelwork to have minimum 2No. coats of high build zinc phosphate primer.

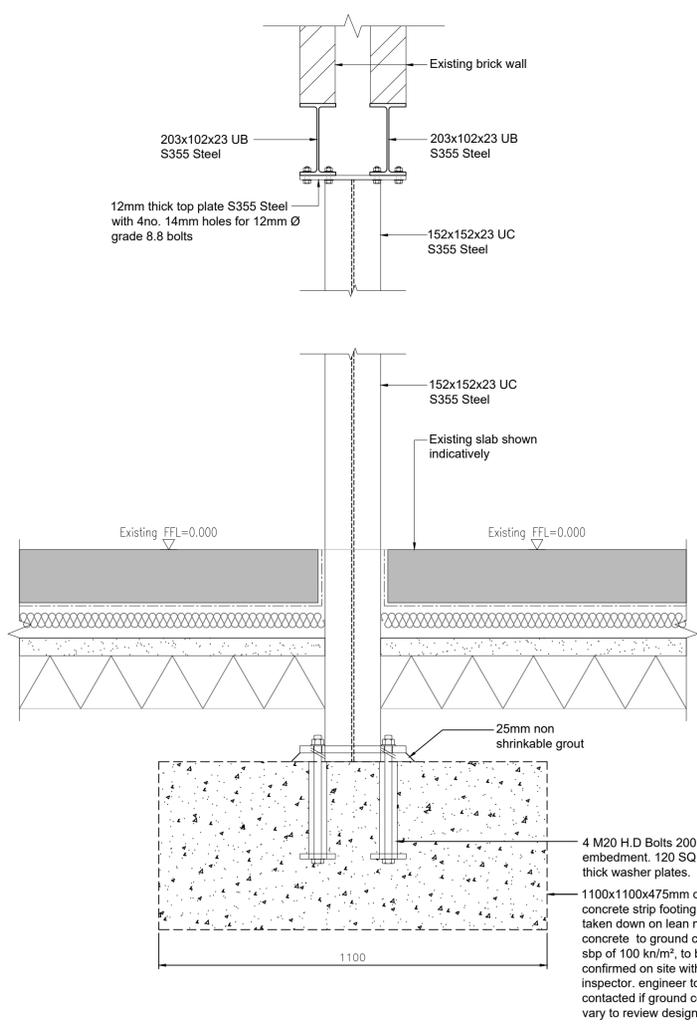All floor timber to have solid timber noggins at maximum 1800mm centers

RIBA
Chartered Practice

Design Summary

scale bar 1:50

0   .5m 1m 1.5m 2m 2.5m 3m 3.5m 4m 4.5m 5m

Existing brick wall

203x102x23 UB
S355 Steel

203x102x23 UB
S355 Steel

12mm thick top plate S355 Steel
with 4no. 14mm holes for 12mm Ø
grade 8.8 bolts

152x152x23 UC
S355 Steel

152x152x23 UC
S355 Steel

Existing slab shown
indicatively

Existing FFL=0.000

Existing FFL=0.000

25mm non
shrinkable grout

4 M20 H.D Bolts 200mm
embedment. 120 SQ x 15mm
thick washer plates.

1100x1100x475mm deep C35
concrete strip footing to be
taken down on lean mix C20
concrete to ground capable of a
sbp of 100 kn/m², to be
confirmed on site with building
inspector. engineer to be
contacted if ground conditions
vary to review design.

1100

Section A-A Through Column (1:10)

PROPOSED GROUND FLOOR PLAN

900x900x450mm dp
concrete pad foundation with
2 layers of A393 mesh at top
and bottom (40mm cover)

178x102x19 UB S355 Steel

152x152x30 UC post with 10mm
cap plate with 4No. M16 bolts.

12mm base plate + 4 No M16
resin holding down bolts at each
end.

152x152x23 UC post with
10mm cap plate with 4No.
M16 bolts.

12mm base plate + 4 No
M16 resin holding down
bolts at each end.

MK9 203x203x46 UC S355 Steel

MK9 203x203x46 UC S355 Steel with 10mm end
plates + 4No. M20 for fixing to proposed columns
(Refer to calculations)

203x203x60 UC S355 Steel with 10mm end plates +
4No. M20 for fixing to proposed columns
(Refer to calculations)

MK6-203x102x23 UB S355 Steel

MK7-203x102x23 UB S355 Steel

152x152x23 UC
S355 Steel
Proposed Column

MK5-178x102x19 UB
S355 Steel

MK4-178x102x19 UB
S355 Steel

Simple connection
detail A

Simple connection
detail B

178x102x19 UB
S355 Steel

2No 178x102x19 UB

MK8-305x305x97 UC S355 Steel with 10mm end plates +
4No. M20 for fixing to proposed columns
(Refer to calculations)

10mm bottom plate to underside of beam to suit
specialist door fixing requirements

203x203x60 UC S355 Steel with 10mm end plates +
4No. M20 for fixing to proposed columns
(Refer to calculations)

1100x1100x475mm deep C35
concrete strip footing to be taken
down on lean mix C20 concrete
to ground capable of a sbp of 100
kN/m², to be confirmed on site
with building inspector. engineer
to be contacted if ground
conditions vary to review design.

203x102x23 UB S355 Steel
(10mm bottom plate to
underside of beam to suit
specialist door fixing
requirements)

152x152x30 UC post with 10mm
cap plate with 4No. M16 bolts.

12mm base plate + 4 No M16
resin holding down bolts at each
end.

MK2-203x102x23 UB S355 Steel

MK3-Naylor R6 Lintel

MK1-203x102x23 UB S355 Steel

Naylor R6 Lintel

152x152x23 UC post with 10mm
cap plate with 4No. M16 bolts.

12mm base plate + 4 No M16
resin holding down bolts at each
end.

1100x1100x475mm deep C35
concrete strip footing to be
taken down on lean mix C20
concrete to ground capable of
a sbp of 100 kN/m², to be
confirmed on site with building
inspector. engineer to be
contacted if ground conditions
vary to review design.

STUDY

W.C.

up

ENTRANCE HALL

rwp
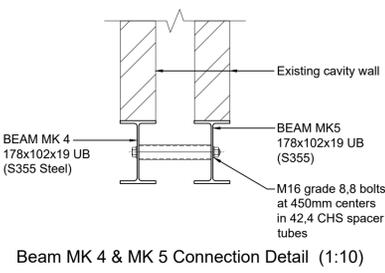
rwp

rwp

rwp

svp

svp

svp

svp

KEY

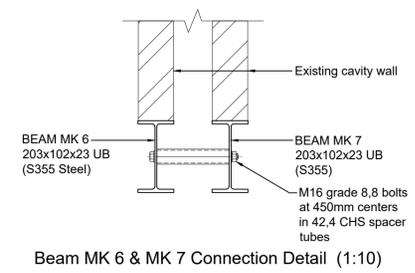| | Min. 215x100x140mm deep C30 concrete padstone |
	Min. 440x100x140mm deep C30 concrete padstone
	Span of treated timber beams to form sun shading area
	50x150 C16 timber rafters @ 400mm ctrs
	Assumed span of existing first floor joists. If found to be different, engineer to be contacted to review design
	Span of existing rafters
	The Contractor shall design, install and maintain all necessary temporary works and shall advise the Architect before commencement of to works, of his proposals for temporary supports and sequence of construction.

Beams to have minimum bearing of
100mm perpendicular to a wall and
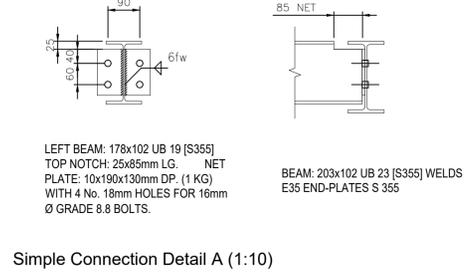150mm in the plane of a wall

Existing cavity wall

BEAM MK 4
178x102x19 UB
(S355 Steel)

BEAM MK5
178x102x19 UB
(S355)

M16 grade 8,8 bolts
at 450mm centers
in 42,4 CHS spacer
tubes

Beam MK 4 & MK 5 Connection Detail (1:10)

Existing cavity wall

BEAM MK 6
203x102x23 UB
(S355 Steel)

BEAM MK 7
203x102x23 UB
(S355)

M16 grade 8,8 bolts
at 450mm centers
in 42,4 CHS spacer
tubes

Beam MK 6 & MK 7 Connection Detail (1:10)

90

6fw

85 NET

LEFT BEAM: 178x102 UB 19 [S355]
TOP NOTCH: 25x85mm LG.   NET
PLATE: 10x190x130mm DP. (1 KG)
WITH 4 No. 18mm HOLES FOR 16mm
Ø GRADE 8.8 BOLTS.

BEAM: 203x102 UB 23 [S355] WELDS
E35 END-PLATES S 355

Simple Connection Detail A (1:10)

90

6fw

85 NET

LEFT BEAM: 178x102 UB 19 [S355]
TOP NOTCH: 25x85mm
BTM NOTCH: 25x85mmLG.   NET
PLATE: 10x190x130mm DP. (1 KG)
WITH 4 No. 18mm HOLES FOR 16mm
Ø GRADE 8.8 BOLTS.

BEAM: 178x102 UB 19 [S355] WELDS
E35 END-PLATES S 355

Simple Connection Detail B (1:10)

LARK
a r c h i t e c t s

Superstructure Design

RIBA
Chartered Practice

LARK Architects Ltd
Summit House, Riparian Way,
The Crossings, Cross Hills,
Keighley, BD20 7BW

Tel: 01535630772
Email: info@larkarchitects.co.uk
Web : www.larkarchitects.co.uk

Job no. :

Page :

By :

Date :

Checked :

LOADING SHEET

EXISTING ROOF $S_k W$.

gk · CLAY PANTILES.   0.6
    · BATTENS + FELT.   0.1
    · RAFTERS.   0.15
    · SERVICES.   0.1
    · PLASTERBOARD.   0.15
                                                                   ——————
                                                                   1.1 kN/m²

qk · SNOW LOAD .   0.6 kN/m²

EXISTING EXTERNAL WALL

gk · 100mm BRICK   2.2
    · 100mm BLOCK + PLASTER   2.35
                                                                   ——————
                                                                   4.55 kN/m²

FIRST FLOOR.

gk · BOARDS + JOISTS.   0.35
    · SOFFIT   0.15
    · SERVICES.   0.1
    · PLASTERBOARD.   0.15
                                                                   ——————
                                                                   0.75 kN/m²

qk · IMPOSED LOADING.   1.5
    · PARTITIONS.   0.5
                                                                   ——————
                                                                   2.0 kN/m²

Job no. :	
Page :	
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Date :	
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BEAM MK1 - CLEAR SPAN 2.15m S355 STEEL.

2.15m

(1.20).
5kN.

LOAD LOADOUT.
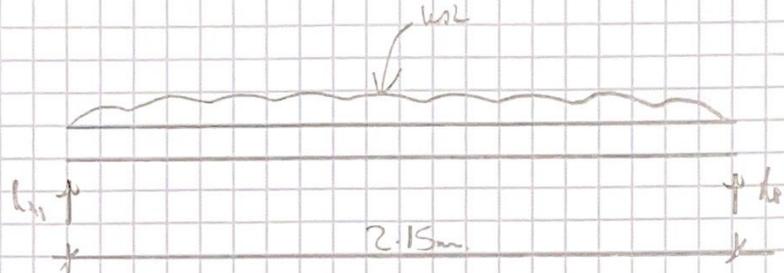
gk
- 100mm BRICK WALL + PLASTER (2.35w/m²) x HEIGHT 2.6m.   6.1
- ROOF (1.1w/m²) x LOADED WIDTH 4.0m/2.   2.2
- FIRST FLOOR (0.75w/m²) x LOADED WIDTH 3.3m/2   1.75.

9.55w/m

qk
- ROOF (0.6w/m²) x LOADED WIDTH 4.0m/2.   1.2
- FIRST FLOOR (2.0w/m²) x 3.3m/2   3.3

4.5w/m

BEAM MK2 - CLEAR SPAN 2.15m S355 STEEL.

gk
- 100mm BRICK + PLASTERBOARD (2.35w/m²) x HEIGHT 2.6m.   6.1
- ROOF (1.1w/m²) x LOADED WIDTH 4.0m/2.   2.2
- FIRST FLOOR (0.75w/m²) x LOADED WIDTH 2.1m/2.   0.8.

9.1w/m

qk
- ROOF (0.6w/m²) x LOADED WIDTH 4.0m/2   1.2
- FIRST FLOOR (2.0w/m²) x WIDTH 2.1m/2   2.1

3.3w/m

	Project		Job Ref.	
Tekla®Tedds	Westwick Lock House (House)		1103	
Lark Architects	Section		Sheet no./rev.	
Unit 10	Mr & Mrs Copnall		1	
Riparian Court	Calc. by	Date	Chk'd by	Date	App'd by	Date
The Crossings, Cross Hills	CJM	07/11/2022				

MK1 STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

Load Envelope - Combination 1

Bending Moment Envelope

Shear Force Envelope

Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead full UDL 9.55 kN/m
	Imposed full UDL 4.5 kN/m
	Dead self weight of beam $\times$ 1

Load combinations

Load combination 1	Support A	Dead $\times$ 1.40
		Imposed $\times$ 1.60
		Dead $\times$ 1.40
		Imposed $\times$ 1.60
	Support B	Dead $\times$ 1.40
		Imposed $\times$ 1.60

	Project		Job Ref.
Tekla.Tedds	Westwick Lock House (House)		1103
Lark Architects	Section		Sheet no./rev.
Unit 10	Mr & Mrs Copnall		2
Riparian Court	Calc. by	Date	Chk'd by	Date	App'd by	Date
The Crossings, Cross Hills	CJM	07/11/2022				

Analysis results

Maximum moment;	M_{max} = **28.4** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **34.5** kN;	V_{min} = **-34.5** kN
Deflection;	δ_{max} = **5.1** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **34.5** kN;	R_{A_min} = **34.5** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **16.1** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **7.4** kN	
Maximum reaction at support B;	R_{B_max} = **34.5** kN;	R_{B_min} = **34.5** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **16.1** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **7.4** kN	

Section details

Section type;   **UKB 203x102x23 (Tata Steel Advance)**

Steel grade;   **S355**

From table 9: Design strength p_y

Thickness of element;   max(T, t) = **9.3** mm

Design strength; p_y = **355** N/mm^2

Modulus of elasticity;   E = **205000** N/mm^2

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.00**;

                                                            $K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section;   d = **169.4** mm

d / t = 35.6 $\times$ ε <= 80 $\times$ ε;   Class 1 plastic

Outstand flanges - Table 11

Width of section;   b = B / 2 = **50.9** mm

b / T = 6.2 $\times$ ε <= 9 $\times$ ε;   Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; F_v = max(abs(V_{max}), abs(V_{min})) = **34.5** kN

d / t < 70 $\times$ ε

Web does not need to be checked for shear buckling

	Project		Job Ref.	
Tekla Tedds	Westwick Lock House (House)		1103	
Lark Architects	Section		Sheet no./rev.	
Unit 10	Mr & Mrs Copnall		3	
Riparian Court	Calc. by	Date	Chk'd by	Date	App'd by	Date
The Crossings, Cross Hills	CJM	07/11/2022				

Shear area; $A_v = t \times D = $ **1097** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v = $ **233.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = max(abs(M_{s1_max}), abs(M_{s1_min})) = $ **28.4** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = $ **83.1** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} = $ **3300** mm

Slenderness ratio; $\lambda = L_E / r_{yy} = $ **139.775**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = $ **0.888**

Torsional index; $x = $ **22.460**

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = $ **0.764**

Ratio - cl.4.3.6.9; $\beta_W = $ **1.000**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = $ **94.821**

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = $ **30.198**

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = $ **7.0**

Perry factor; $\eta_{LT} = max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = $ **0.452**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = $ **225** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = $ **340.9** N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = $ **150.3** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = $ **21.3** kNm

Moment at centre-line of segment; $M_3 = $ **28.4** kNm

Moment at three quarter point of segment; $M_4 = $ **21.3** kNm

Maximum moment in segment; $M_{abs} = $ **28.4** kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = $ **28.4** kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = $ **0.925**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = $ **35.2** kNm

$M_b / m_{LT} = $ **38** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 = $ **9.167** mm

Maximum deflection span 1; $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = $ **5.109** mm

*PASS - Maximum deflection does not exceed deflection limit*

;

	Project		Job Ref.	
Tekla Tedds	Westwick Lock House (House)		1103	
Lark Architects	Section		Sheet no./rev.	
Unit 10	Mr & Mrs Copnall		4	
Riparian Court	Calc. by	Date	Chk'd by	Date	App'd by	Date
The Crossings, Cross Hills	CJM	07/11/2022				

MK2 STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

Load Envelope - Combination 1

Bending Moment Envelope

Shear Force Envelope
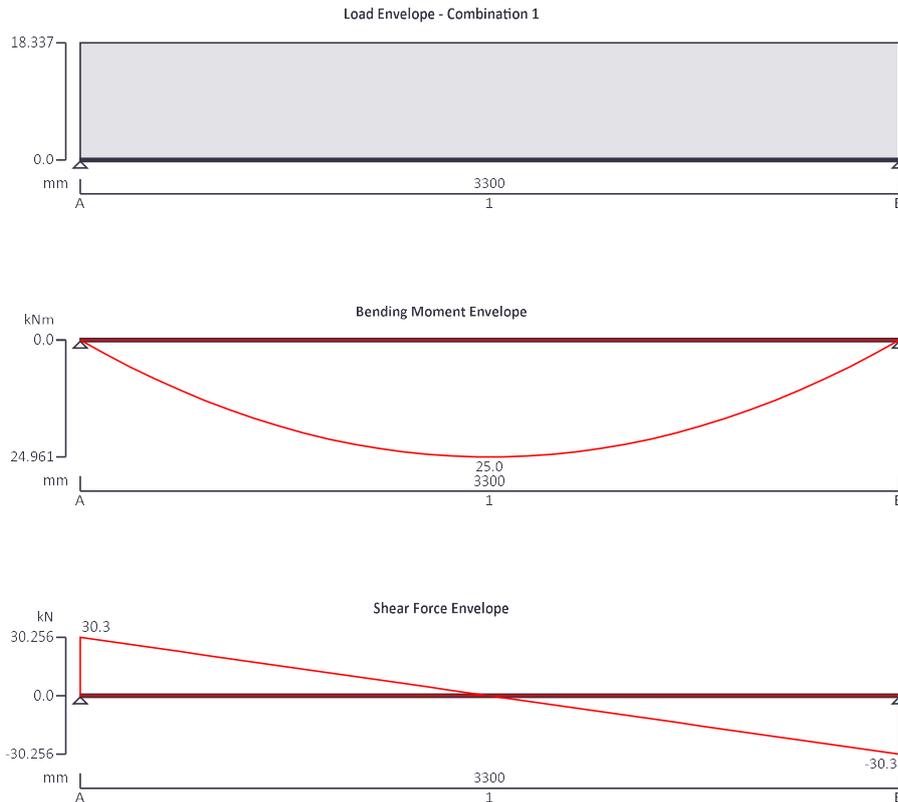
Support conditions

Support A
Vertically restrained
Rotationally free

Support B
Vertically restrained
Rotationally free

Applied loading

Beam loads
Dead full UDL 9.1 kN/m
Imposed full UDL 3.3 kN/m
Dead self weight of beam × 1

Load combinations

Load combination 1
Support A
Dead × 1.40
Imposed × 1.60
Dead × 1.40
Imposed × 1.60

Support B
Dead × 1.40
Imposed × 1.60

	Project		Job Ref.	
Tekla. Tedds	Westwick Lock House (House)		1103	
Lark Architects	Section		Sheet no./rev.	
Unit 10	Mr & Mrs Copnall		5	
Riparian Court	Calc. by	Date	Chk'd by	Date	App'd by	Date
The Crossings, Cross Hills	CJM	07/11/2022				

Analysis results

Maximum moment;	M_{max} = **25** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **30.3** kN;	V_{min} = **-30.3** kN
Deflection;	δ_{max} = **4.5** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **30.3** kN;	R_{A_min} = **30.3** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **15.4** kN
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **5.4** kN
Maximum reaction at support B;	R_{B_max} = **30.3** kN;	R_{B_min} = **30.3** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **15.4** kN
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **5.4** kN

Section details

Section type;   **UKB 203x102x23 (Tata Steel Advance)**

Steel grade;   **S355**

From table 9: Design strength p_y

Thickness of element;   max(T, t) = **9.3** mm

Design strength; p_y = **355** N/mm^2

Modulus of elasticity;   E = **205000** N/mm^2

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.00**;

$K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section;   d = **169.4** mm

d / t = 35.6 $\times$ ε <= 80 $\times$ ε;   Class 1 plastic

Outstand flanges - Table 11

Width of section;   b = B / 2 = **50.9** mm

b / T = 6.2 $\times$ ε <= 9 $\times$ ε;   Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; F_v = max(abs(V_{max}), abs(V_{min})) = **30.3** kN

d / t < 70 $\times$ ε

Web does not need to be checked for shear buckling

	Project		Job Ref.	
Tekla.Tedds	Westwick Lock House (House)		1103	
Lark Architects	Section		Sheet no./rev.	
Unit 10	Mr & Mrs Copnall		6	
Riparian Court	Calc. by	Date	Chk'd by	Date	App'd by	Date
The Crossings, Cross Hills	CJM	07/11/2022				

Shear area; $A_v = t \times D = $ **1097** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v = $ **233.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(abs(M_{s1_max}), abs(M_{s1_min})) = $ **25** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = $ **83.1** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} = $ **3300** mm

Slenderness ratio; $\lambda = L_E / r_{yy} = $ **139.775**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = $ **0.888**

Torsional index; $x = $ **22.460**

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = $ **0.764**

Ratio - cl.4.3.6.9; $\beta_W = $ **1.000**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = $ **94.821**

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = $ **30.198**

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = $ **7.0**

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = $ **0.452**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = $ **225** N/mm^2

    $\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = $ **340.9** N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = $ **150.3** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = $ **18.7** kNm

Moment at centre-line of segment; $M_3 = $ **25** kNm

Moment at three quarter point of segment; $M_4 = $ **18.7** kNm

Maximum moment in segment; $M_{abs} = $ **25** kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = $ **25** kNm

Equivalent uniform moment factor for lateral-torsional buckling;

    $m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = $ **0.925**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = $ **35.2** kNm

    $M_b / m_{LT} = $ **38** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 = $ **9.167** mm

Maximum deflection span 1; $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = $ **4.518** mm

PASS - Maximum deflection does not exceed deflection limit

;

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MK3 - CLEAR SPAN 1·6m CONCRETE LINTEL

WDL LOADING
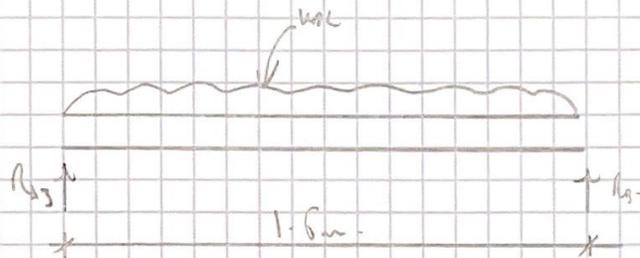
SAME LOADING AS BEAM MK1

$$g_k = 9.55 \text{ kN/m}$$
$$q_k = 4.5 \text{ kN/m}$$
$$\underline{ \text{ kN/m}}$$

NAYLOR RC LINTEL FOR 1·6 CLEAR SPAN = 14·4 kN/m

14·05 kN/m < 14·4 kN/m

∴ PROVIDE NAYLOR RC CONCRETE LINTEL

Hi-Spec & Fair Faced Ranges

All units are prestressed to ensure optimum performance and come with 30 minutes fire rating as standard. With a range of 18 different sections sizes and with lengths available up to 4800mm on certain sections sizes.

Hi-Spec Finish
An ex-steel mould finish and made from wet cast concrete. May see small air holes on the surface, aggregate and concrete fines to bottom arrises. For use normally in plastered/ situations. **Not for use in exposed painted situations.**

Fair Faced Finish
Type C Fair Faced Finish. All corners, arrises and faces on the lintel are perfect. For use in exposed/painted situations.
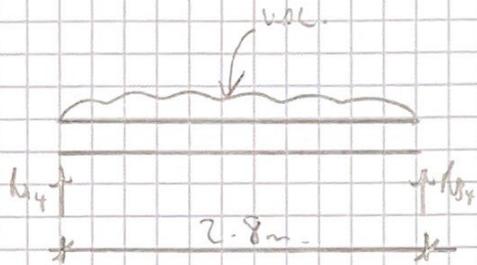
Order/Specification Code
Use the Hi-Spec reference on our load Tables (e.g. S5). For Fair Faced Lintels preceed the lintel reference with the word 'Faced', ie Faced S5.

Hi-Spec Range		P100	S4	R6	R9	R12
Load Table Units suitable for 100mm wide walls		65 / 100	100 / 100	140 / 100	215 / 100	290 / 100
Fire Resistance Available (mins)		30	30	30	30	30
Suitable for Foundation Use		yes	yes	yes	yes	yes
Maximum Stock Length Available		2400mm	3000mm	3600mm	3600mm	3600mm
					Longer lengths available on request - up to 4800mm	
Available Range Finish			Faced	Faced	Faced	Faced
UNFACTORED LOADS IN kN/m						
Length	Clear Span	100x65	100x100	100x140	100x215	100x290
900mm	700mm	12.97	18.00	48.90	78.18	100.05
1100mm	900mm	7.96	14.20	39.00	62.44	79.90
1200mm	1000mm	6.47	12.80	34.23	56.72	72.57
1500mm	1200mm	4.50	11.25	24.38	48.57	60.85
1800mm	1500mm	2.86	7.25	15.96	36.27	49.65
2100mm	1800mm	1.95	5.02	11.21	25.78	41.91
2400mm	2100mm	1.21	3.66	8.27	19.21	31.70
2700mm	2400mm	n/a	2.77	6.32	14.83	24.53
3000mm	2700mm	n/a	2.15	4.97	11.76	19.49
3300mm	3000mm	n/a	n/a	3.99	9.53	15.83
3600mm	3200mm	n/a	n/a	3.48	8.36	13.49
Lintel Weight kg/m		16	23	34	53	70

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BEAM MK4 - CLEAR SPAN 2.8m SJSS STEEL

UDL.

R_{A_4} R_{B_4}

2.8m

UDL LOADING:-   SLAB:-

g_k · 100mm BRICK + PLASTER (2.35 w/m²) × HEIGHT 2.6m.   6.1
· ROOF (1.1 w/m²) × LOADED WIDTH 3.0m /2   1.65
· FIRST FLOOR (0.75 w/m²) × LOADED WIDTH 2.1m /2   0.8
                                                      ─────
                                                      8.55 w/m

q_k · ROOF (0.6 w/m²) × LOADED WIDTH 3.0m /2   0.9
· FIRST FLOOR (2.0 w/m²) × 2.1m /2   2.1
                                                      ─────
                                                      3.0 w/m

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The Crossings, Cross Hills	CJM	07/11/2022				

MK4 STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

Load Envelope - Combination 1

Bending Moment Envelope

Shear Force Envelope

Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead full UDL 8.55 kN/m

Imposed full UDL 3 kN/m

Dead self weight of beam $\times$ 1

Load combinations

Load combination 1

Support A

Dead $\times$ 1.40

Imposed $\times$ 1.60

Dead $\times$ 1.40

Imposed $\times$ 1.60

Support B

Dead $\times$ 1.40

Imposed $\times$ 1.60

	Project		Job Ref.
Tekla Tedds	Westwick Lock House (House)		1103
Lark Architects	Section		Sheet no./rev.
Unit 10	Mr & Mrs Copnall		8
Riparian Court	Calc. by	Date	Chk'd by	Date	App'd by	Date
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Analysis results

Maximum moment;	M_{max} = **16.7** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **23.8** kN;	V_{min} = **-23.8** kN
Deflection;	δ_{max} = **3.4** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **23.8** kN;	R_{A_min} = **23.8** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **12.2** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **4.2** kN	
Maximum reaction at support B;	R_{B_max} = **23.8** kN;	R_{B_min} = **23.8** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **12.2** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **4.2** kN	

Section details

Section type; **UKB 178x102x19 (Tata Steel Advance)**

Steel grade; **S355**

From table 9: Design strength p_y

Thickness of element; max(T, t) = **7.9** mm

Design strength; p_y = **355** N/mm^2

Modulus of elasticity; E = **205000** N/mm^2

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.00**;

  $K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

  ε = $\sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section; d = **146.8** mm

  d / t = 34.7 × ε <= 80 × ε; Class 1 plastic

Outstand flanges - Table 11

Width of section; b = B / 2 = **50.6** mm

  b / T = 7.3 × ε <= 9 × ε; Class 1 plastic

  Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; F_v = max(abs(V_{max}), abs(V_{min})) = **23.8** kN

  d / t < 70 × ε

  Web does not need to be checked for shear buckling

Shear area; A_v = t × D = **853** mm^2

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The Crossings, Cross Hills	CJM	07/11/2022				

Design shear resistance; $P_v = 0.6 \times p_y \times A_v = \textbf{181.8 kN}$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = max(abs(M_{s1\_max}), abs(M_{s1\_min})) = \textbf{16.7 kNm}$

Moment capacity low shear - cl.4.2.5.2; $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \textbf{60.8 kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} = \textbf{2800 mm}$

Slenderness ratio; $\lambda = L_E / r_{yy} = \textbf{117.956}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = \textbf{0.888}$

Torsional index; $x = \textbf{22.560}$

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \textbf{0.806}$

Ratio - cl.4.3.6.9; $\beta_W = \textbf{1.000}$

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \textbf{84.415}$

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \textbf{30.198}$

$\lambda_{LT} > \lambda_{L0}$ *- Allowance should be made for lateral-torsional buckling*

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = \textbf{7.0}$

Perry factor; $\eta_{LT} = max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \textbf{0.380}$

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = \textbf{283.9 N/mm}^2$

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \textbf{373.3 N/mm}^2$

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \textbf{176.9 N/mm}^2$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = \textbf{12.5 kNm}$

Moment at centre-line of segment; $M_3 = \textbf{16.7 kNm}$

Moment at three quarter point of segment; $M_4 = \textbf{12.5 kNm}$

Maximum moment in segment; $M_{abs} = \textbf{16.7 kNm}$

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \textbf{16.7 kNm}$

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \textbf{0.925}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = \textbf{30.3 kNm}$

$M_b / m_{LT} = \textbf{32.8 kNm}$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 = \textbf{7.778 mm}$

Maximum deflection span 1; $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = \textbf{3.379 mm}$

PASS - Maximum deflection does not exceed deflection limit

;

BEAM MK5 - CLEAR SPAN 2.8m S355 STEEL

UDL LOADING.

SKDN.

g^k · 100mm BRICK + PLASTER ... $(2.35 kN/m^2) \times 2.6m$ —   6.1.
· ROOF $(1.1 kN/m^2) \times$ WIDTH $4.3m /2$.   2.4.

8.5 kN/m.

q^k · ROOF $(0.6 kN/m^2) \times$ WIDTH $4.3m /2$ 1.3 kN/m.

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Lark Architects	Section		Sheet no./rev.	
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The Crossings, Cross Hills	CJM	07/11/2022				

MK5 STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

Load Envelope - Combination 1

Bending Moment Envelope

Shear Force Envelope
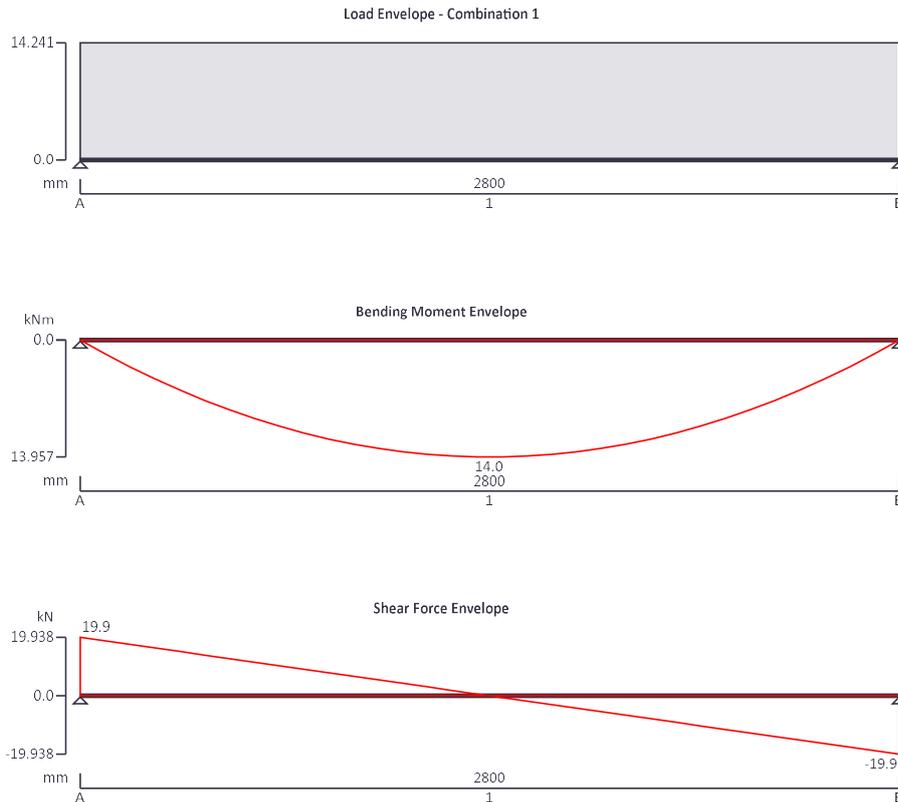
Support conditions

Support A — Vertically restrained
Rotationally free

Support B — Vertically restrained
Rotationally free

Applied loading

Beam loads — Dead full UDL 8.5 kN/m
Imposed full UDL 1.3 kN/m
Dead self weight of beam $\times$ 1

Load combinations

Load combination 1 — Support A — Dead $\times$ 1.40
Imposed $\times$ 1.60
Dead $\times$ 1.40
Imposed $\times$ 1.60

Support B — Dead $\times$ 1.40
Imposed $\times$ 1.60

	Project			Job Ref.	
Tekla Tedds	Westwick Lock House (House)			1103	
Lark Architects Unit 10 Riparian Court The Crossings, Cross Hills	Section			Sheet no./rev.	
	Mr & Mrs Copnall			11	
	Calc. by CJM	Date 07/11/2022	Chk'd by	Date	App'd by	Date

Analysis results

Maximum moment;	M_{max} = **14** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **19.9** kN;	V_{min} = **-19.9** kN
Deflection;	δ_{max} = **2.9** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **19.9** kN;	R_{A_min} = **19.9** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **12.2** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **1.8** kN	
Maximum reaction at support B;	R_{B_max} = **19.9** kN;	R_{B_min} = **19.9** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **12.2** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **1.8** kN	

Section details

Section type;   **UKB 178x102x19 (Tata Steel Advance)**

Steel grade;   **S355**

From table 9: Design strength p_y

Thickness of element;   max(T, t) = **7.9** mm

Design strength; p_y = **355** N/mm²

Modulus of elasticity;   E = **205000** N/mm²

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.00**;

$K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section;   d = **146.8** mm

d / t = 34.7 × ε <= 80 × ε;   Class 1 plastic

Outstand flanges - Table 11

Width of section;   b = B / 2 = **50.6** mm

b / T = 7.3 × ε <= 9 × ε;   Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; F_v = max(abs(V_{max}), abs(V_{min})) = **19.9** kN

d / t < 70 × ε

Web does not need to be checked for shear buckling

Shear area; A_v = t × D = **853** mm²

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The Crossings, Cross Hills	CJM	07/11/2022				

Design shear resistance; $P_v = 0.6 \times p_y \times A_v = \textbf{181.8 kN}$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \textbf{14 kNm}$

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \textbf{60.8 kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} = \textbf{2800 mm}$

Slenderness ratio; $\lambda = L_E / r_{yy} = \textbf{117.956}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = \textbf{0.888}$

Torsional index; $x = \textbf{22.560}$

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \textbf{0.806}$

Ratio - cl.4.3.6.9; $\beta_W = \textbf{1.000}$

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \textbf{84.415}$

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \textbf{30.198}$

$\boldsymbol{\lambda_{LT} > \lambda_{L0}}$ *- Allowance should be made for lateral-torsional buckling*

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = \textbf{7.0}$

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \textbf{0.380}$

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = \textbf{283.9 N/mm}^2$

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \textbf{373.3 N/mm}^2$

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \textbf{176.9 N/mm}^2$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = \textbf{10.5 kNm}$

Moment at centre-line of segment; $M_3 = \textbf{14 kNm}$

Moment at three quarter point of segment; $M_4 = \textbf{10.5 kNm}$

Maximum moment in segment; $M_{abs} = \textbf{14 kNm}$

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \textbf{14 kNm}$

Equivalent uniform moment factor for lateral-torsional buckling;

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \textbf{0.925}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = \textbf{30.3 kNm}$

$M_b / m_{LT} = \textbf{32.8 kNm}$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 = \textbf{7.778 mm}$

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \textbf{2.875 mm}$

PASS - Maximum deflection does not exceed deflection limit

;

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BEAM MK6 - SPAN 4.95m S355 STEEL

MK4   MK5
$g_k = 12.2kN$ $g_k = 12.2kN$
$q_k = 4.2kN$ $q_k = 1.8kN$

UDL

| 1.85m | 3.30m |

4.95m

WALL LOADING.   SELF.

g_k : 100mm BRICK + PLASTER $(2.35kN/m^2) \times 2.6m$ $6.1kN/m$

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The Crossings, Cross Hills	CJM	07/11/2022				

MK6 STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

Load Envelope - Combination 1

Bending Moment Envelope

Shear Force Envelope

Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free
Support C	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead point load 24.4 kN at 1650 mm
	Imposed point load 6 kN at 1650 mm
	Dead full UDL 6.1 kN/m
	Dead self weight of beam $\times$ 1

Load combinations

Load combination 1	Support A	Dead $\times$ 1.40
		Imposed $\times$ 1.60
		Dead $\times$ 1.40
		Imposed $\times$ 1.60
	Support B	Dead $\times$ 1.40

	Project			Job Ref.	
Tekla Tedds	Westwick Lock House (House)			**1103**	
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The Crossings, Cross Hills	**CJM**	**07/11/2022**				

Imposed × 1.60

Dead × 1.40

Imposed × 1.60

Support C   Dead × 1.40

Imposed × 1.60

Analysis results

Maximum moment;	M_{max} = **8.2** kNm;	M_{min} = **-9.3** kNm
Maximum moment span 1;	M_{s1_max} = **0.2** kNm;	M_{s1_min} = **-9.3** kNm
Maximum moment span 2;	M_{s2_max} = **8.2** kNm;	M_{s2_min} = **-9.3** kNm
Maximum shear;	V_{max} = **17.6** kN;	V_{min} = **-13** kN
Maximum shear span 1;	V_{s1_max} = **1.7** kN;	V_{s1_min} = **-13** kN
Maximum shear span 2;	V_{s2_max} = **17.6** kN;	V_{s2_min} = **-12.1** kN
Deflection;	δ_{max} = **1.3** mm;	δ_{min} = **0.1** mm
Deflection span 1;	δ_{s1_max} = **0** mm;	δ_{s1_min} = **0.1** mm
Deflection span 2;	δ_{s2_max} = **1.3** mm;	δ_{s2_min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **1.7** kN;	R_{A_min} = **1.7** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **1.2** kN	
Maximum reaction at support B;	R_{B_max} = **74.3** kN;	R_{B_min} = **74.3** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **46.2** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **6** kN	
Maximum reaction at support C;	R_{C_max} = **12.1** kN;	R_{C_min} = **12.1** kN
Unfactored dead load reaction at support C;	R_{C_Dead} = **8.6** kN	

Section details

Section type;   **UKB 203x102x23 (Tata Steel Advance)**

Steel grade;   **S355**

From table 9: Design strength p_y

Thickness of element;   max(T, t) = **9.3** mm

Design strength; p_y = **355** N/mm^2

Modulus of elasticity;   E = **205000** N/mm^2
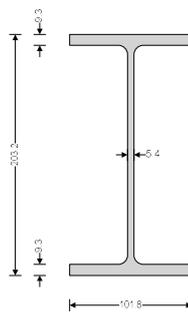
Lateral restraint

Span 1 has lateral restraint at supports only

Span 2 has full lateral restraint

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.00**;

$K_{LT.B}$ = **1.00**;

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$K_{LT.C}$ = **1.00**;

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section;   d = **169.4** mm

d / t = 35.6 × ε <= 80 × ε;   Class 1 plastic

Outstand flanges - Table 11

Width of section;   b = B / 2 = **50.9** mm

b / T = 6.2 × ε <= 9 × ε;   Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; F_v = max(abs(V_{max}), abs(V_{min})) = **17.6** kN

d / t < 70 × ε

Web does not need to be checked for shear buckling

Shear area; A_v = t × D = **1097** mm^2

Design shear resistance; P_v = 0.6 × p_y × A_v = **233.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 - Section 4.2.5

Design bending moment;   M = max(abs(M_{s1_max}), abs(M_{s1_min})) = **9.3** kNm

Moment capacity low shear - cl.4.2.5.2; M_c = min(p_y × S_{xx}, 1.5 × p_y × Z_{xx}) = **83.1** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; L_E = 1.0 × L_{s1} = **1650** mm

Slenderness ratio; λ = L_E / r_{yy} = **69.888**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter;   u = **0.888**

Torsional index;   x = **22.460**

Slenderness factor;   v = 1 / $[1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.906**

Ratio - cl.4.3.6.9; β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7; λ_{LT} = u × v × λ × $\sqrt{[\beta_W]}$ = **56.230**

Limiting slenderness - Annex B.2.2; λ_{L0} = 0.4 × $(\pi^2 \times E / p_y)^{0.5}$ = **30.198**

λ_{LT} > λ_{L0} - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; α_{LT} = **7.0**

Perry factor; η_{LT} = max(α_{LT} × (λ_{LT} - λ_{L0}) / 1000, 0) = **0.182**

Euler stress; p_E = π^2 × E / λ_{LT}^2 = **639.9** N/mm^2

ϕ_{LT} = (p_y + (η_{LT} + 1) × p_E) / 2 = **555.8** N/mm^2

Bending strength - Annex B.2.1; p_b = p_E × p_y / (ϕ_{LT} + (ϕ_{LT}^2 - p_E × p_y)$^{0.5}$) = **269.9** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; M_2 = **0.1** kNm

Moment at centre-line of segment; M_3 = **1.6** kNm

Moment at three quarter point of segment; M_4 = **4.7** kNm

Maximum moment in segment; M_{abs} = **9.3** kNm

Maximum moment governing buckling resistance; M_{LT} = M_{abs} = **9.3** kNm

Equivalent uniform moment factor for lateral-torsional buckling;

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$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \textbf{0.440}$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = \textbf{63.2}$ kNm

$M_b / m_{LT} = \textbf{143.6}$ kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s2} / 360 = \textbf{9.306}$ mm

Maximum deflection span 2; $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = \textbf{1.34}$ mm

PASS - Maximum deflection does not exceed deflection limit

;

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Checked :	

BEAM MK7 - SPAN 4.95m S355 STEEL.

UDL LOADING.   SERV.

gk • 100mm BRICK + PLASTER (2.35 w/m²) × LOADED HEIGHT 2.6m   6.1
    • RWF (1.1 w/m²) × LOADED WIDTH 2.1m /2   1.15.
    • FIRST FLOOR (0.75 w/m²) × LOADED WIDTH 2.7m /2   1.0.

                                                                   8.25 kN/m

qk • ROOF (0.6 w/m²) × LOADED WIDTH 2.1m /2   0.65
    • FIRST FLOOR (2.0 w/m²) × LOADED WIDTH 2.7m /2   2.7

                                                                   3.35 kN/m

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MK7 STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

Load Envelope - Combination 1

Bending Moment Envelope

Shear Force Envelope
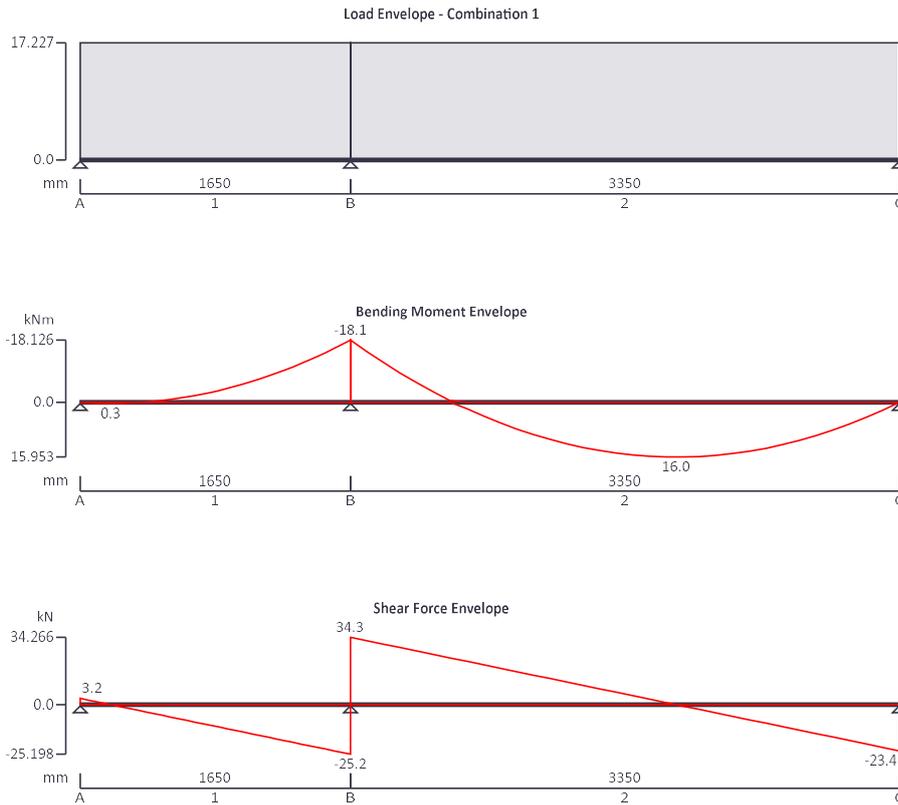
Support conditions

Support A
Vertically restrained
Rotationally free

Support B
Vertically restrained
Rotationally free

Support C
Vertically restrained
Rotationally free

Applied loading

Beam loads
Imposed full UDL 3.35 kN/m
Dead full UDL 8.25 kN/m
Dead self weight of beam $\times$ 1

Load combinations

Load combination 1
Support A
Dead $\times$ 1.40
Imposed $\times$ 1.60
Dead $\times$ 1.40
Imposed $\times$ 1.60

Support B
Dead $\times$ 1.40
Imposed $\times$ 1.60

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Dead × 1.40

Imposed × 1.60

Support C

Dead × 1.40

Imposed × 1.60

Analysis results

Maximum moment;	M_{max} = **16** kNm;	M_{min} = **-18.1** kNm
Maximum moment span 1;	M_{s1_max} = **0.3** kNm;	M_{s1_min} = **-18.1** kNm
Maximum moment span 2;	M_{s2_max} = **16** kNm;	M_{s2_min} = **-18.1** kNm
Maximum shear;	V_{max} = **34.3** kN;	V_{min} = **-25.2** kN
Maximum shear span 1;	V_{s1_max} = **3.2** kN;	V_{s1_min} = **-25.2** kN
Maximum shear span 2;	V_{s2_max} = **34.3** kN;	V_{s2_min} = **-23.4** kN
Deflection;	δ_{max} = **2.5** mm;	δ_{min} = **0.3** mm
Deflection span 1;	δ_{s1_max} = **0** mm;	δ_{s1_min} = **0.3** mm
Deflection span 2;	δ_{s2_max} = **2.5** mm;	δ_{s2_min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **3.2** kN;	R_{A_min} = **3.2** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **1.6** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **0.6** kN	
Maximum reaction at support B;	R_{B_max} = **59.5** kN;	R_{B_min} = **59.5** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **29.3** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **11.6** kN	
Maximum reaction at support C;	R_{C_max} = **23.4** kN;	R_{C_min} = **23.4** kN
Unfactored dead load reaction at support C;	R_{C_Dead} = **11.5** kN	
Unfactored imposed load reaction at support C;	$R_{C\_Imposed}$ = **4.6** kN	

Section details

Section type; **UKB 203x102x23 (Tata Steel Advance)**

Steel grade; **S355**

From table 9: Design strength p_y

Thickness of element; max(T, t) = **9.3** mm

Design strength; p_y = **355** N/mm²

Modulus of elasticity; E = **205000** N/mm²

Lateral restraint

Span 1 has lateral restraint at supports only

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Span 2 has full lateral restraint

Effective length factors

Effective length factor in major axis;	K_x = **1.00**
Effective length factor in minor axis;	K_y = **1.00**
Effective length factor for lateral-torsional buckling;	$K_{LT.A}$ = **1.00**;
	$K_{LT.B}$ = **1.00**;
	$K_{LT.C}$ = **1.00**;

Classification of cross sections - Section 3.5

$\epsilon = \sqrt{[275\ N/mm^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section;   d = **169.4** mm

d / t = 35.6 × ε <= 80 × ε;   Class 1 plastic

Outstand flanges - Table 11

Width of section;   b = B / 2 = **50.9** mm

b / T = 6.2 × ε <= 9 × ε;   Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; F_v = max(abs(V_{max}), abs(V_{min})) = **34.3** kN

d / t < 70 × ε

Web does not need to be checked for shear buckling

Shear area; A_v = t × D = **1097** mm²

Design shear resistance; P_v = 0.6 × p_y × A_v = **233.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 - Section 4.2.5

Design bending moment;   M = max(abs(M_{s1_max}), abs(M_{s1_min})) = **18.1** kNm

Moment capacity low shear - cl.4.2.5.2; M_c = min(p_y × S_{xx}, 1.5 × p_y × Z_{xx}) = **83.1** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; L_E = 1.0 × L_{s1} = **1650** mm

Slenderness ratio;   λ = L_E / r_{yy} = **69.888**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter;   u = **0.888**

Torsional index;   x = **22.460**

Slenderness factor;   v = 1 / [1 + 0.05 × (λ / x)²]$^{0.25}$ = **0.906**

Ratio - cl.4.3.6.9; β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7; λ_{LT} = u × v × λ × $\sqrt{[\beta_W]}$ = **56.230**

Limiting slenderness - Annex B.2.2; λ_{L0} = 0.4 × (π² × E / p_y)$^{0.5}$ = **30.198**

λ_{LT} > λ_{L0} - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; α_{LT} = **7.0**

Perry factor; η_{LT} = max(α_{LT} × (λ_{LT} - λ_{L0}) / 1000, 0) = **0.182**

Euler stress; p_E = π² × E / λ_{LT}^2 = **639.9** N/mm²

ϕ_{LT} = (p_y + (η_{LT} + 1) × p_E) / 2 = **555.8** N/mm²

Bending strength - Annex B.2.1; p_b = p_E × p_y / (ϕ_{LT} + (ϕ_{LT}^2 - p_E × p_y)$^{0.5}$) = **269.9** N/mm²

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Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = \textbf{0.1}$ kNm

Moment at centre-line of segment; $M_3 = \textbf{3.2}$ kNm

Moment at three quarter point of segment; $M_4 = \textbf{9.2}$ kNm

Maximum moment in segment; $M_{abs} = \textbf{18.1}$ kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = \textbf{18.1}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$$m_{LT} = max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \textbf{0.440}$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{xx} = \textbf{63.2}$ kNm

$M_b / m_{LT} = \textbf{143.6}$ kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s2} / 360 = \textbf{9.306}$ mm

Maximum deflection span 2; $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = \textbf{2.505}$ mm

PASS - Maximum deflection does not exceed deflection limit

;

PADSTONE DESIGNS.

BEAM MK1 = 22.4 kN

100mm BRICK, F_k = 3.5 N/mm²

FIND ALLOWABLE BEARING TO BRICK.

COS = 1.25 × F_k /3.5.

COS = 1.25 N/mm²

FIND BEARING AREA = R_1/COS.

$$\frac{22.4 \, kN \times 10^3}{1.25 \, N/mm^2} = 17920 \, mm^2$$

17920 mm² / 100mm MIN BEARING. = 179.2 mm LONG.

∴ PROVIDE A 215mm × 100mm × 150mm DEEP C30 CONCRETE PADSTONE

BEAM MK2 PADSTONE DESIGN.

NEWTON = 19.7 kN

$$\frac{19.7 \times 10^3}{1.25 \, N/mm^2} = 15760 \, mm^2 \quad /100mm \, MIN.$$

= 157.6 mm LONG.

∴ PROVIDE 215mm × 100mm × 150mm DEEP C30 CONCRETE PADSTONE.

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PADSTONE DESIGN CONTINUED.

MK4 = 23.8kN.

$$\frac{23.8kN \times 10^3}{1.25} = 23800 \, mm^2 \quad / \, 100mm \; MIN \; BEARING.$$

= 190.4 mm LONG.

∴ PROVIDE 215mm x 100mm x 150mm DEEP C30 CONCRETE PADSTONE.

MK5 = 19.9kN

$$\frac{19.9kN \times 10^3}{1.25} = 15920 \, mm^2 \quad / \, 100mm \; BEARING$$

= 159.2 mm LONG.

∴ PROVIDE 159.2mm LONG C30 CONCRETE PADSTONE.

BEAM MK6 = 17.6kN

$$\frac{17.6kN \times 10^3}{1.25} = 14080 \, mm^2 \quad / \, 100mm \; BEARING.$$

= 140.8mm LONG.

∴ PROVIDE 215mm x 100mm x 150mm DEEP C30 CONCRETE PADSTONE

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STEEL MEMBER DESIGN (BS5950)

STEEL MEMBER DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

Section details

Section type;   **UC 152x152x23 (BS4-1)**

Steel grade;   **S355**

From table 9: Design strength p_y

Thickness of element; $max(T, t) = $ **6.8** mm

Design strength; $p_y = $ **355** N/mm²

Modulus of elasticity; $E = $ **205000** N/mm²
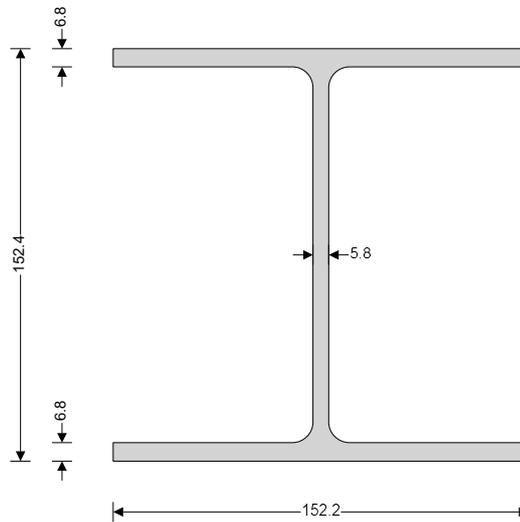
Lateral restraint

Distance between major axis restraints; $L_x = $ **2500** mm

Distance between minor axis restraints; $L_y = $ **0** mm

Effective length factors

Effective length factor in major axis; $K_x = $ **1.00**

Effective length factor in minor axis; $K_y = $ **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT} = $ **1.00**;

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = $ **0.88**

Internal compression parts - Table 11

Depth of section; $d = $ **123.6** mm

Stress ratios; $r1 = min(F_c / (d \times t \times p_{yw}), 1) = $ **0.526**

$r2 = F_c / (A \times p_{yw}) = $ **0.129**

$d / t = 24.2 \times \varepsilon <= max(80 \times \varepsilon / (1 + r1), 40 \times \varepsilon)$;   Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = $ **76.1** mm

$b / T = 12.7 \times \varepsilon <= 15 \times \varepsilon$;   Class 3 semi-compact

Section is class 3 semi-compact

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Compression members - Section 4.7

Design compression force; F_c = **133.8** kN

Effective length for major (x-x) axis buckling - Section 4.7.3

Effective length for buckling; L_{Ex} = $L_x \times K_x$ = **2500** mm

Slenderness ratio - cl.4.7.2; λ_x = L_{Ex} / r_{xx} = **38.243**

Compressive strength - Section 4.7.5

Limiting slenderness; λ_0 = $0.2 \times (\pi^2 \times E / p_y)^{0.5}$ = **15.099**

Strut curve - Table 23; b

Robertson constant; α_x = **3.5**

Perry factor; η_x = $\alpha_x \times (\lambda_x - \lambda_0) / 1000$ = **0.081**

Euler stress; p_{Ex} = $\pi^2 \times E / \lambda_x^2$ = **1383.4** N/mm²

 ϕ_x = $(p_y + (\eta_x + 1) \times p_{Ex}) / 2$ = **925.2** N/mm²

Compressive strength - Annex C.1; p_{cx} = $p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5})$ = **321.1** N/mm²

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4; P_{cx} = $A \times p_{cx}$ = **939.1** kN

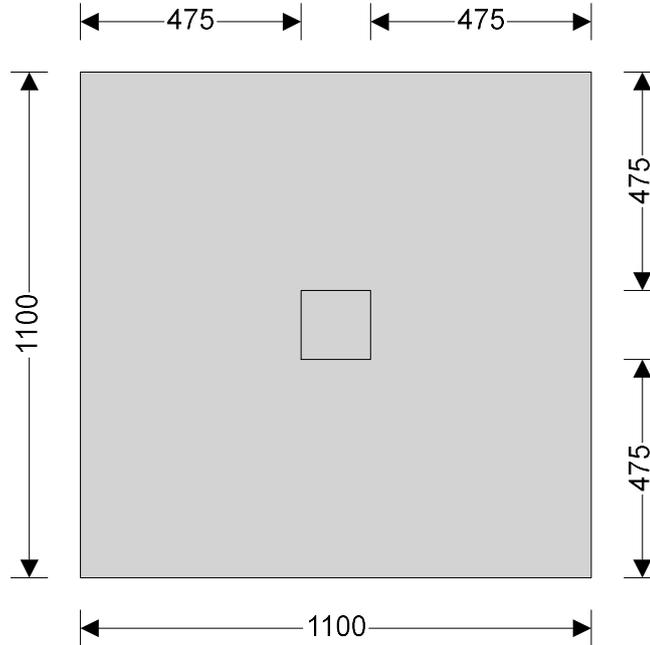***PASS - Compression resistance exceeds design compression force***

;

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PAD FOOTING ANALYSIS & DESIGN (BS8110)

PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07

Pad footing details

Length of pad footing;	L = **1100** mm
Width of pad footing;	B = **1100** mm
Area of pad footing;	$A = L \times B$ = **1.210** m^2
Depth of pad footing;	h = **475** mm
Depth of soil over pad footing;	h_{soil} = **300** mm
Density of concrete;	ρ_{conc} = **23.6** kN/m^3

Column details

Column base length;	l_A = **150** mm
Column base width;	b_A = **150** mm
Column eccentricity in x;	e_{PxA} = **0** mm
Column eccentricity in y;	e_{PyA} = **0** mm

Soil details

Density of soil;	ρ_{soil} = **20.0** kN/m^3
Design shear strength;	ϕ' = **25.0** deg
Design base friction;	δ = **19.3** deg
Allowable bearing pressure;	$P_{bearing}$ = **100** kN/m^2

Axial loading on column

Dead axial load on column;	P_{GA} = **52.2** kN
Imposed axial load on column;	P_{QA} = **40.9** kN
Wind axial load on column;	P_{WA} = **0.0** kN
Total axial load on column;	P_A = **93.1** kN

Foundation loads

Dead surcharge load;	F_{Gsur} = **2.500** kN/m^2

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Imposed surcharge load;	$F_{Qsur} = \mathbf{1.500}$ kN/m^2
Pad footing self weight;	$F_{swt} = h \times \rho_{conc} = \mathbf{11.210}$ kN/m^2
Soil self weight;	$F_{soil} = h_{soil} \times \rho_{soil} = \mathbf{6.000}$ kN/m^2
Total foundation load;	$F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = \mathbf{25.7}$ kN

Calculate pad base reaction

Total base reaction;	$T = F + P_A = \mathbf{118.8}$ kN
Eccentricity of base reaction in x;	$e_{Tx} = (P_A \times e_{PxA} + M_{xA} + H_{xA} \times h) / T = \mathbf{0}$ mm
Eccentricity of base reaction in y;	$e_{Ty} = (P_A \times e_{PyA} + M_{yA} + H_{yA} \times h) / T = \mathbf{0}$ mm

Check pad base reaction eccentricity

$$abs(e_{Tx}) / L + abs(e_{Ty}) / B = \mathbf{0.000}$$

Base reaction acts within middle third of base

Calculate pad base pressures

$$q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = \mathbf{98.152}\ kN/m^2$$
$$q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = \mathbf{98.152}\ kN/m^2$$
$$q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = \mathbf{98.152}\ kN/m^2$$
$$q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = \mathbf{98.152}\ kN/m^2$$

Minimum base pressure;	$q_{min} = min(q_1, q_2, q_3, q_4) = \mathbf{98.152}$ kN/m^2
Maximum base pressure;	$q_{max} = max(q_1, q_2, q_3, q_4) = \mathbf{98.152}$ kN/m^2

PASS - Maximum base pressure is less than allowable bearing pressure

98.2 kN/m^2 98.2 kN/m^2

98.2 kN/m^2 98.2 kN/m^2

Material details

Characteristic strength of concrete;	$f_{cu} = \mathbf{35}$ N/mm^2

Calculate pad lengths in x direction

Left hand length;	$L_L = L / 2 + e_{PxA} = \mathbf{550}$ mm
Right hand length;	$L_R = L / 2 - e_{PxA} = \mathbf{550}$ mm

Calculate rate of change of base pressure in x direction

Length of base reaction;	$L_x = L = \mathbf{1100}$ mm

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Rate of change of base pressure; $C_x = [(q_1 + q_2) - (q_3 + q_4)] / (2 \times L_x) = \textbf{0.000}$ kN/m²/m

Calculate pad lengths in y direction

Top length; $L_T = B / 2 - e_{PyA} = \textbf{550}$ mm

Bottom length; $L_B = B / 2 + e_{PyA} = \textbf{550}$ mm

Calculate rate of change of base pressure in y direction

Length of base reaction; $L_y = B = \textbf{1100}$ mm

Rate of change of base pressure; $C_y = [(q_1 + q_3) - (q_2 + q_4)] / (2 \times L_y) = \textbf{0.000}$ kN/m²/m

Calculate minimum depth of unreinforced pad footing

Average pressure to left of pad footing; $q_L = (q_1 + q_2) / 2 - C_x \times (L_L - l_A / 2) / 2 = \textbf{98.152}$ kN/m²

Minimum depth to left of pad footing; $h_{Lmin} = (L_L-l_A/2) \times max(0.15 \times [(q_L/1kN/m^2)^2 /(f_{cu}/1N/mm^2)]^{1/4},1) = \textbf{475}$ mm

Average pressure to right of pad footing; $q_R = (q_3 + q_4) / 2 - C_x \times (L_R - l_A / 2) / 2 = \textbf{98.152}$ kN/m²

Minimum depth to right of pad footing; $h_{Rmin} = (L_R-l_A/2) \times max(0.15 \times [(q_R/1kN/m^2)^2 /(f_{cu}/1N/mm^2)]^{1/4},1) = \textbf{475}$ mm

Average pressure to top of pad footing; $q_T = (q_2 + q_4) / 2 - C_y \times (L_T - b_A / 2) / 2 = \textbf{98.152}$ kN/m²

Minimum depth to top of pad footing; $h_{Tmin} = (L_T-b_A/2) \times max(0.15 \times [(q_T/1kN/m^2)^2 /(f_{cu}/1N/mm^2)]^{1/4},1) = \textbf{475}$ mm

Average pressure to bottom of pad footing; $q_B = (q_1 + q_3) / 2 - C_y \times (L_B - b_A / 2) / 2 = \textbf{98.152}$ kN/m²

Minimum depth to bottom of pad footing; $h_{Bmin} = (L_B-b_A/2) \times max(0.15 \times [(q_B/1kN/m^2)^2 /(f_{cu}/1N/mm^2)]^{1/4},1) = \textbf{475}$ mm

Minimum depth of unreinforced pad footing; $h_{min} = max(h_{Lmin}, h_{Rmin}, h_{Tmin}, h_{Bmin}, 300 mm) = \textbf{475}$ mm

PASS - Unreinforced pad footing depth is greater than minimum

;

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Job no. :

Page :

By :

Date :

Checked :

BEAM MK8 - CLEAR SPAN 7.5m MILD S355 STEEL.

UDL

7.5m

UDL WORKING.

SERV.

g/k
- EXTERNAL WALL (4.55w/m²) × HEIGHT 2.6m.   11.85.
- EXISTING ROOF (1.1w/m²) × 1.5m /2   0.85.
- FIRST FLOOR (0.75w/m²) × 2.7m /2.   1.0.
- PROPOSED ROOF (0.6w/m²) × 2.3m /2.   0.7

14.4w/m

q/k
- EXISTING ROOF (1.0w/m²) × 1.5m/2.   0.75
- FIRST FLOOR (2.0w/m²) × 2.7m/2   2.7
- PROPOSED ROOF (0.6w/m²) × 2.3/2   0.7

4.15w/m

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MK8 STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

Load Envelope - Combination 1

Bending Moment Envelope

Shear Force Envelope

Support conditions

Support A   Vertically restrained
                         Rotationally free

Support B   Vertically restrained
                         Rotationally free

Applied loading

Beam loads   Dead full UDL 14.4 kN/m
                         Imposed full UDL 4.15 kN/m
                         Dead self weight of beam $\times$ 1

Load combinations

Load combination 1   Support A   Dead $\times$ 1.40
                                          Imposed $\times$ 1.60
                                          Dead $\times$ 1.40
                                          Imposed $\times$ 1.60
                         Support B   Dead $\times$ 1.40
                                          Imposed $\times$ 1.60

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Analysis results

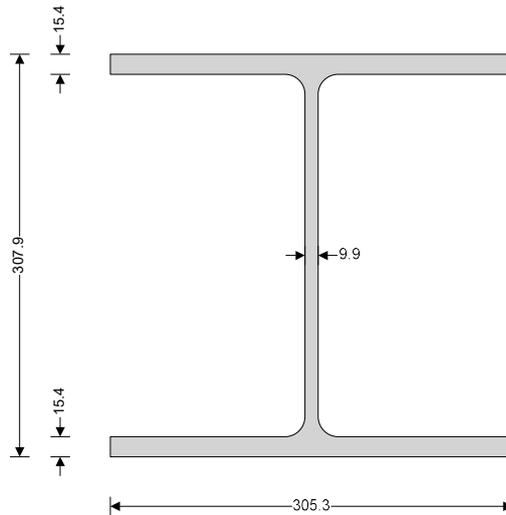|			
Maximum moment;	M_{max} = **197.8** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **105.5** kN;	V_{min} = **-105.5** kN
Deflection;	δ_{max} = **17.6** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **105.5** kN;	R_{A_min} = **105.5** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **57.6** kN
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **15.6** kN
Maximum reaction at support B;	R_{B_max} = **105.5** kN;	R_{B_min} = **105.5** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **57.6** kN
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **15.6** kN

Section details

Section type;   **UC 305x305x97 (BS4-1)**

Steel grade;   **S355**

From table 9: Design strength p_y

Thickness of element;   max(T, t) = **15.4** mm

Design strength; p_y = **355** N/mm^2

Modulus of elasticity;   E = **205000** N/mm^2

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.00**;

$K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

ε = $\sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Internal compression parts - Table 11

Depth of section;   d = **246.7** mm

d / t = 28.3 × ε <= 80 × ε;   Class 1 plastic

Outstand flanges - Table 11

Width of section;   b = B / 2 = **152.7** mm

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$b / T = 11.3 \times \varepsilon <= 15 \times \varepsilon;$ Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = max(abs(V_{max}), abs(V_{min})) = $ **105.5** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D = $ **3048** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v = $ **649.3** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = max(abs(M_{s1_max}), abs(M_{s1_min})) = $ **197.8** kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange; $\beta_{2f} = 10 \times \varepsilon = $ **8.801**

Limiting value for class 3 semi-compact flange; $\beta_{3f} = 15 \times \varepsilon = $ **13.202**

Limiting value for class 2 compact web; $\beta_{2w} = 100 \times \varepsilon = $ **88.014**

Limiting value for class 3 semi-compact web; $\beta_{3w} = 120 \times \varepsilon = $ **105.617**

Effective plastic modulus - cl.3.5.6.2

$\quad S_{eff} = min(Z_{xx} + (S_{xx} - Z_{xx}) \times min([((\beta_{3w} / (d / t))^2 - 1) / ((\beta_{3w} / \beta_{2w})^2 - 1)], [(\beta_{3f} / (b / T) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S_{xx}) = $ **1542693** mm^3

Moment capacity low shear - cl.4.2.5.2; $M_c = min(p_y \times S_{eff}, 1.2 \times p_y \times Z_{xx}) = $ **547.7** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} = $ **7500** mm

Slenderness ratio; $\lambda = L_E / r_{yy} = $ **97.480**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = $ **0.850**

Torsional index; $x = $ **19.257**

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = $ **0.814**

Ratio - cl.4.3.6.9; $\beta_W = S_{eff} / S_{xx} = $ **0.969**

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = $ **66.374**

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = $ **30.198**

$\lambda_{LT} > \lambda_{L0}$ *- Allowance should be made for lateral-torsional buckling*

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = $ **7.0**

Perry factor; $\eta_{LT} = max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = $ **0.253**

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = $ **459.3** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = $ **465.3** N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = $ **234.1** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = $ **148.3** kNm

Moment at centre-line of segment; $M_3 = $ **197.8** kNm

Moment at three quarter point of segment; $M_4 = $ **148.3** kNm

Maximum moment in segment; $M_{abs} = $ **197.8** kNm

Maximum moment governing buckling resistance; $M_{LT} = M_{abs} = $ **197.8** kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$\quad m_{LT} = max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = $ **0.925**

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Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{eff} = $ **361.1** kNm

$M_b / m_{LT} = $ **390.4** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360 = $ **20.833** mm

Maximum deflection span 1; $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = $ **17.614** mm

PASS - Maximum deflection does not exceed deflection limit

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Tekla Tedds	Westwick Lock House (House)			1103	
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MK8 COLUMN STEEL MEMBER DESIGN (BS5950)

STEEL MEMBER DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

Section details

Section type;   **UC 152x152x23 (BS4-1)**

Steel grade;   **S355**

From table 9: Design strength p_y

Thickness of element; $\max(T, t) = \textbf{6.8}$ mm

Design strength; $p_y = \textbf{355}$ N/mm^2

Modulus of elasticity; $E = \textbf{205000}$ N/mm^2
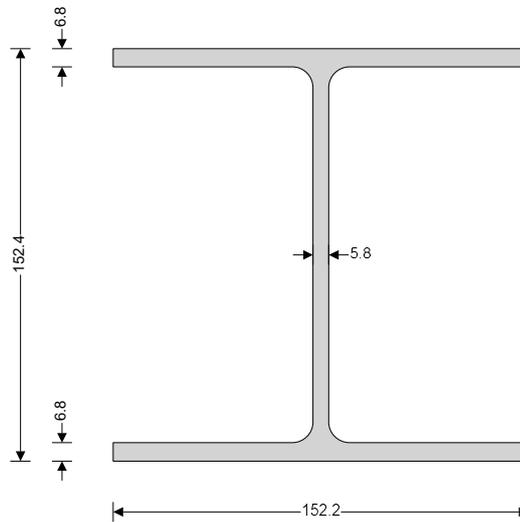
Lateral restraint

Distance between major axis restraints; $L_x = \textbf{3000}$ mm

Distance between minor axis restraints; $L_y = \textbf{0}$ mm

Effective length factors

Effective length factor in major axis; $K_x = \textbf{1.00}$

Effective length factor in minor axis; $K_y = \textbf{1.00}$

Effective length factor for lateral-torsional buckling; $K_{LT} = \textbf{1.00}$;

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = \textbf{0.88}$

Internal compression parts - Table 11

Depth of section; $d = \textbf{123.6}$ mm

Stress ratios; $r1 = \min(F_c / (d \times t \times p_{yw}), 1) = \textbf{0.415}$

$r2 = F_c / (A \times p_{yw}) = \textbf{0.102}$

$d / t = 24.2 \times \varepsilon <= \max(80 \times \varepsilon / (1 + r1), 40 \times \varepsilon);$ Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = \textbf{76.1}$ mm

$b / T = 12.7 \times \varepsilon <= 15 \times \varepsilon;$ Class 3 semi-compact

Section is class 3 semi-compact

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Compression members - Section 4.7

Design compression force; F_c = **105.5** kN

Effective length for major (x-x) axis buckling - Section 4.7.3

Effective length for buckling; $L_{Ex} = L_x \times K_x$ = **3000** mm

Slenderness ratio - cl.4.7.2; $\lambda_x = L_{Ex} / r_{xx}$ = **45.891**

Compressive strength - Section 4.7.5

Limiting slenderness; $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5}$ = **15.099**

Strut curve - Table 23;   b

Robertson constant; α_x = **3.5**

Perry factor; $\eta_x = \alpha_x \times (\lambda_x - \lambda_0) / 1000$ = **0.108**

Euler stress; $p_{Ex} = \pi^2 \times E / \lambda_x^2$ = **960.7** N/mm^2

                                                          $\phi_x = (p_y + (\eta_x + 1) \times p_{Ex}) / 2$ = **709.6** N/mm^2

Compressive strength - Annex C.1; $p_{cx} = p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5})$ = **306.5** N/mm^2

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4; $P_{cx} = A \times p_{cx}$ = **896.4** kN

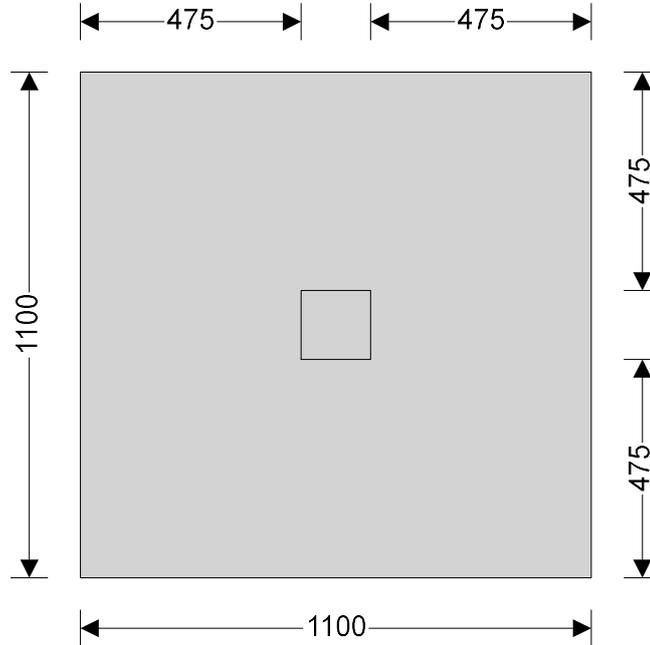*PASS - Compression resistance exceeds design compression force*

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MK8 PAD FOOTING ANALYSIS & DESIGN (BS8110)

PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07

Pad footing details

Length of pad footing;		L = **1100** mm
Width of pad footing;		B = **1100** mm
Area of pad footing;		$A = L \times B$ = **1.210** m^2
Depth of pad footing;		h = **475** mm
Depth of soil over pad footing;		h_{soil} = **300** mm
Density of concrete;		ρ_{conc} = **23.6** kN/m^3

Column details

Column base length;		l_A = **150** mm
Column base width;		b_A = **150** mm
Column eccentricity in x;		e_{PxA} = **0** mm
Column eccentricity in y;		e_{PyA} = **0** mm

Soil details

Density of soil;		ρ_{soil} = **20.0** kN/m^3
Design shear strength;		ϕ' = **25.0** deg
Design base friction;		δ = **19.3** deg
Allowable bearing pressure;		$P_{bearing}$ = **100** kN/m^2

Axial loading on column

Dead axial load on column;		P_{GA} = **57.6** kN
Imposed axial load on column;		P_{QA} = **15.6** kN
Wind axial load on column;		P_{WA} = **0.0** kN
Total axial load on column;		P_A = **73.2** kN

Foundation loads

Dead surcharge load;		F_{Gsur} = **2.500** kN/m^2

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Imposed surcharge load; $F_{Qsur} = $ **1.500** kN/m²

Pad footing self weight; $F_{swt} = h \times \rho_{conc} = $ **11.210** kN/m²

Soil self weight; $F_{soil} = h_{soil} \times \rho_{soil} = $ **6.000** kN/m²

Total foundation load; $F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = $ **25.7** kN

Calculate pad base reaction

Total base reaction; $T = F + P_A = $ **98.9** kN

Eccentricity of base reaction in x; $e_{Tx} = (P_A \times e_{PxA} + M_{xA} + H_{xA} \times h) / T = $ **0** mm

Eccentricity of base reaction in y; $e_{Ty} = (P_A \times e_{PyA} + M_{yA} + H_{yA} \times h) / T = $ **0** mm

Check pad base reaction eccentricity

$abs(e_{Tx}) / L + abs(e_{Ty}) / B = $ **0.000**

Base reaction acts within middle third of base

Calculate pad base pressures

$q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = $ **81.706** kN/m²

$q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = $ **81.706** kN/m²

$q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = $ **81.706** kN/m²

$q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = $ **81.706** kN/m²

Minimum base pressure; $q_{min} = min(q_1, q_2, q_3, q_4) = $ **81.706** kN/m²

Maximum base pressure; $q_{max} = max(q_1, q_2, q_3, q_4) = $ **81.706** kN/m²

PASS - Maximum base pressure is less than allowable bearing pressure

81.7 kN/m²   81.7 kN/m²

81.7 kN/m²   81.7 kN/m²

Material details

Characteristic strength of concrete; $f_{cu} = $ **35** N/mm²

Calculate pad lengths in x direction

Left hand length; $L_L = L / 2 + e_{PxA} = $ **550** mm

Right hand length; $L_R = L / 2 - e_{PxA} = $ **550** mm

Calculate rate of change of base pressure in x direction

Length of base reaction; $L_x = L = $ **1100** mm

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Rate of change of base pressure; $C_x = [(q_1 + q_2) - (q_3 + q_4)] / (2 \times L_x) = \textbf{0.000}$ kN/m²/m

Calculate pad lengths in y direction

Top length; $L_T = B / 2 - e_{PyA} = \textbf{550}$ mm

Bottom length; $L_B = B / 2 + e_{PyA} = \textbf{550}$ mm

Calculate rate of change of base pressure in y direction

Length of base reaction; $L_y = B = \textbf{1100}$ mm

Rate of change of base pressure; $C_y = [(q_1 + q_3) - (q_2 + q_4)] / (2 \times L_y) = \textbf{0.000}$ kN/m²/m

Calculate minimum depth of unreinforced pad footing

Average pressure to left of pad footing; $q_L = (q_1 + q_2) / 2 - C_x \times (L_L - l_A / 2) / 2 = \textbf{81.706}$ kN/m²

Minimum depth to left of pad footing; $h_{Lmin} = (L_L - l_A/2) \times max(0.15 \times [(q_L/1kN/m^2)^2 / (f_{cu}/1N/mm^2)]^{1/4}, 1) = \textbf{475}$ mm

Average pressure to right of pad footing; $q_R = (q_3 + q_4) / 2 - C_x \times (L_R - l_A / 2) / 2 = \textbf{81.706}$ kN/m²

Minimum depth to right of pad footing; $h_{Rmin} = (L_R - l_A/2) \times max(0.15 \times [(q_R/1kN/m^2)^2 / (f_{cu}/1N/mm^2)]^{1/4}, 1) = \textbf{475}$ mm

Average pressure to top of pad footing; $q_T = (q_2 + q_4) / 2 - C_y \times (L_T - b_A / 2) / 2 = \textbf{81.706}$ kN/m²

Minimum depth to top of pad footing; $h_{Tmin} = (L_T - b_A/2) \times max(0.15 \times [(q_T/1kN/m^2)^2 / (f_{cu}/1N/mm^2)]^{1/4}, 1) = \textbf{475}$ mm

Average pressure to bottom of pad footing; $q_B = (q_1 + q_3) / 2 - C_y \times (L_B - b_A / 2) / 2 = \textbf{81.706}$ kN/m²

Minimum depth to bottom of pad footing; $h_{Bmin} = (L_B - b_A/2) \times max(0.15 \times [(q_B/1kN/m^2)^2 / (f_{cu}/1N/mm^2)]^{1/4}, 1) = \textbf{475}$ mm

Minimum depth of unreinforced pad footing; $h_{min} = max(h_{Lmin}, h_{Rmin}, h_{Tmin}, h_{Bmin}, 300 mm) = \textbf{475}$ mm

PASS - Unreinforced pad footing depth is greater than minimum

;

Job no. :

Page :

By :

Date :

Checked :

BEAM MK 9 - CLEAR SPAN 5.2m S355 STEEL.

5.2m

UDL LOADING.   SCW.

gk · ROOF (0.75w/m²) × LOADED WIDTH 2.6m /2.   1.0w/m

qk · ROOF (0.6w/m²) × LOADED WIDTH 2.6m /2   1.3w/m

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Tekla. Tedds	Westwick Lock House (House)		1103	
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MK9 STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

Load Envelope - Combination 1

Bending Moment Envelope

Shear Force Envelope

Support conditions

Support A   Vertically restrained
                                       Rotationally free

Support B   Vertically restrained
                                       Rotationally free

Applied loading

Beam loads   Dead full UDL 1 kN/m
                                       Imposed full UDL 1.3 kN/m
                                       Dead self weight of beam $\times$ 1

	Project		Job Ref.	
Tekla Tedds	Westwick Lock House (House)		1103	
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Load combinations

Load combination 1   Support A   Dead × 1.40

                                                               Imposed × 1.60

                                                               Dead × 1.40

                                                               Imposed × 1.60

                                            Support B   Dead × 1.40

                                                               Imposed × 1.60

Analysis results

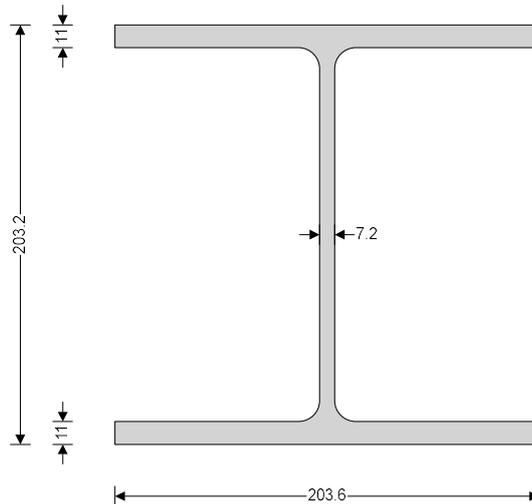| Maximum moment;	M_{max} = **13.9** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **10.7** kN;	V_{min} = **-10.7** kN
Deflection;	δ_{max} = **2.8** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **10.7** kN;	R_{A_min} = **10.7** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **3.8** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **3.4** kN	
Maximum reaction at support B;	R_{B_max} = **10.7** kN;	R_{B_min} = **10.7** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **3.8** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **3.4** kN	

Section details

Section type;   **UC 203x203x46 (BS4-1)**

Steel grade;   **S355**

From table 9: Design strength p$_y$

Thickness of element;   max(T, t) = **11.0** mm

Design strength; p_y = **355** N/mm^2

Modulus of elasticity;   E = **205000** N/mm^2

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.00**;

                                                    $K_{LT.B}$ = **1.00**;

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Classification of cross sections - Section 3.5

$\epsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = \textbf{0.88}$

Internal compression parts - Table 11

Depth of section; $d = \textbf{160.8}$ mm

$d / t = 25.4 \times \epsilon <= 80 \times \epsilon$;   Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = \textbf{101.8}$ mm

$b / T = 10.5 \times \epsilon <= 15 \times \epsilon$;   Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \textbf{10.7}$ kN

$d / t < 70 \times \epsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D = \textbf{1463}$ mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v = \textbf{311.6}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \textbf{13.9}$ kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange; $\beta_{2f} = 10 \times \epsilon = \textbf{8.801}$

Limiting value for class 3 semi-compact flange; $\beta_{3f} = 15 \times \epsilon = \textbf{13.202}$

Limiting value for class 2 compact web; $\beta_{2w} = 100 \times \epsilon = \textbf{88.014}$

Limiting value for class 3 semi-compact web; $\beta_{3w} = 120 \times \epsilon = \textbf{105.617}$

Effective plastic modulus - cl.3.5.6.2

$S_{eff} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \times \min([((\beta_{3w} / (d / t))^2 - 1) / ((\beta_{3w} / \beta_{2w})^2 - 1)], [(\beta_{3f} / (b / T) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S_{xx}) = \textbf{490411}$ mm^3

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z_{xx}) = \textbf{174.1}$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} = \textbf{5200}$ mm

Slenderness ratio; $\lambda = L_E / r_{yy} = \textbf{101.280}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = \textbf{0.847}$

Torsional index; $x = \textbf{17.713}$

Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \textbf{0.785}$

Ratio - cl.4.3.6.9; $\beta_W = S_{eff} / S_{xx} = \textbf{0.986}$

Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = \textbf{66.816}$

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \textbf{30.198}$

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = \textbf{7.0}$

Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \textbf{0.256}$

Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = \textbf{453.2}$ N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \textbf{462.2}$ N/mm^2

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \textbf{232.6}$ N/mm^2

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Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; M_2 = **10.4** kNm

Moment at centre-line of segment; M_3 = **13.9** kNm

Moment at three quarter point of segment; M_4 = **10.4** kNm

Maximum moment in segment; M_{abs} = **13.9** kNm

Maximum moment governing buckling resistance; M_{LT} = M_{abs} = **13.9** kNm

Equivalent uniform moment factor for lateral-torsional buckling;

$$m_{LT} = max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \textbf{0.925}$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{eff}$ = **114** kNm

$$M_b / m_{LT} = \textbf{123.3} \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection; δ_{lim} = L_{s1} / 360 = **14.444** mm

Maximum deflection span 1; δ = max(abs(δ_{max}), abs(δ_{min})) = **2.798** mm

PASS - Maximum deflection does not exceed deflection limit

;

PORTAL FRAME

gk = 1.4w
qk = 1.7w

mk9.
gk = 3.8w
qk = 3.4w

B1
W2

W1.   C1   C2   2.5m

8.0m
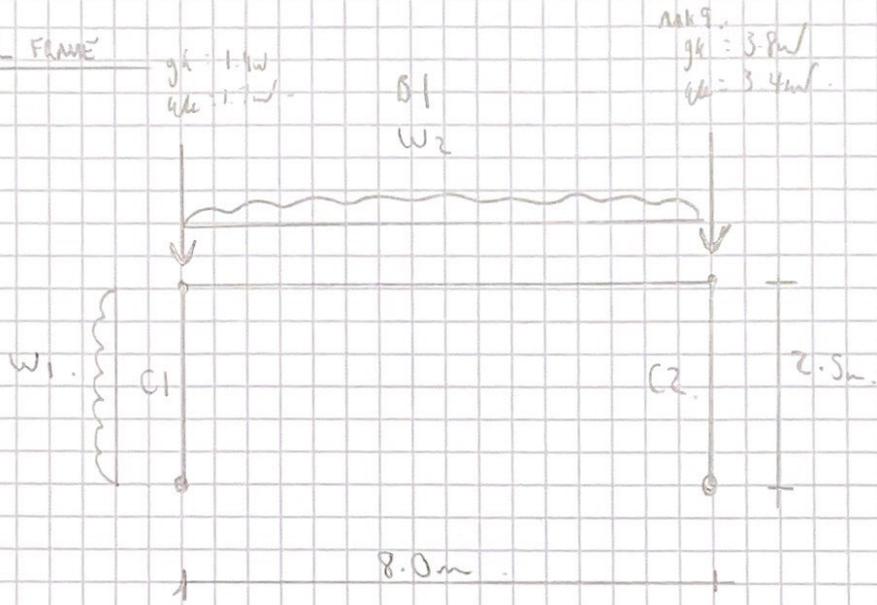
W1

$1.0w/m^2 \times 10.5m / 2 = 5.25 w/m.$

W2

gk · $(0.65w/m^2) \times 2.5m / 2 = 0.8 w/m.$

qk · $0.6w/m^2) \times 2.5m / 2 = 0.75 w/m$

W3

$1.0w/m^2 \times 14.0m / 2 = 7.0 w/m$

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STEEL 2D ANALYSIS & DESIGN (EN1993)

STEEL MEMBER ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 4.4.08

ANALYSIS

Tedds calculation version 1.0.37
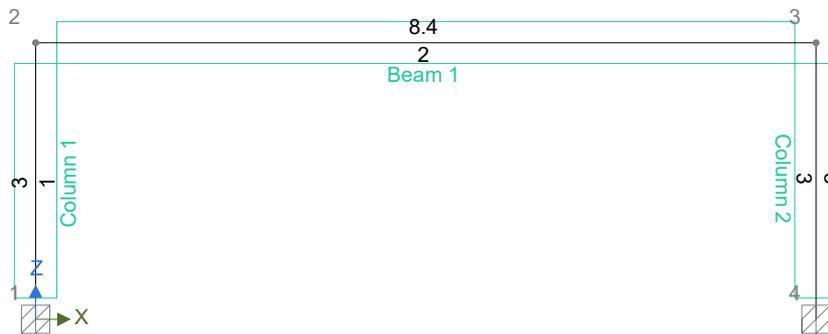
Geometry

Geometry (m) - Steel (EC3)

Materials

Name	Density	Youngs Modulus	Shear Modulus	Thermal Coefficient
	(kg/m^3)	kN/mm^2	kN/mm^2	°C^{-1}
Steel (EC3)	7850	210	80.8	0.000012

Sections

Name	Area	Moment of inertia		Shear area parallel to	
		Major	Minor	Minor	Major
	(cm^2)	(cm^4)	(cm^4)	(cm^2)	(cm^2)
SHS 120x120x5.0	22.7	497.7	497.7	11.4	11.4
UC 203x203x60	76.4	6124.5	2064.6	19.7	52.6
UC 152x152x30	38.3	1748	560.5	10.2	25.9
UC 152x152x30 1	38.3	1748	560.5	10.2	25.9

Nodes

Node	Co-ordinates		Freedom			Coordinate system		Spring		
	X	Z	X	Z	Rot.	Name	Angle	X	Z	Rot.
	(m)	(m)					(°)	(kN/m)	(kN/m)	kNm/°
1	0	0	Fixed	Fixed	Fixed		0	0	0	0
2	0	3	Free	Free	Free		0	0	0	0
3	8.4	3	Free	Free	Free		0	0	0	0
4	8.4	0	Fixed	Fixed	Fixed		0	0	0	0

Elements

Element	Length	Nodes		Section	Material	Releases			Rotated
	(m)	Start	End			Start moment	End moment	Axial	
1	3	1	2	UC 152x152x30	Steel (EC3)	Fixed	Fixed	Fixed	

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Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
2	8.4	2	3	UC 203x203x60	Steel (EC3)	Fixed	Fixed	Fixed	
3	3	3	4	UC 152x152x30 1	Steel (EC3)	Fixed	Fixed	Fixed	

Members

Name	Elements	
	Start	End
Column 1	1	1
Beam 1	2	2
Column 2	3	3

Loading

Self weight included

Load combination factors

Load combination	Self Weight	Permanent	Snow	Wind
$1.35G + 1.5\psi_0Q + 1.5S$ (Strength)	1.35	1.35	1.50	
$1.35G + 1.5Q + 1.5\psi_0S + 1.5\psi_0W$ (Strength)	1.35	1.35	0.75	0.75
$1.0G + 1.0\psi_0Q + 1.0S + 0.5W$ (Service)	1.00	1.00	1.00	0.50
$1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$ (Strength)	1.35	1.35	0.75	1.50
$1.0G + 1.0\psi_0Q + 0.5S + 1.0W$ (Service)	1.00	1.00	0.50	1.00
$1.0G + 1.5W$ (Strength)	1.00	1.00		1.50
$1.0G + 1.0W$ (Service)	1.00	1.00		1.00

Node loads

Node	Load case	Force		Moment
		X (kN)	Z (kN)	(kNm)
2	Permanent	0	1.9	0
3	Permanent	0	3.8	0
2	Snow	0	1.7	0
3	Snow	0	3.4	0

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam 1	Permanent	UDL	GlobalZ	0.8 kN/m
Column 1	Wind	UDL	GlobalZ	5.25 kN/m
Column 2	Wind	UDL	GlobalZ	7 kN/m

Results

Node deflections

Load combination: $1.35G + 1.5\psi_0Q + 1.5S$ (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X (mm)	Z (mm)	(°)	
1	0	0	0	

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Node	Deflection		Rotation	Co-ordinate system
	X	Z		
	(mm)	(mm)	(°)	
2	0	0.1	0.08169	
3	0	0.1	-0.08146	
4	0	0	0	

Load combination: 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X	Z		
	(mm)	(mm)	(°)	
1	0	0	0	
2	0	0.1	0.08171	
3	0	0.1	-0.08145	
4	0	0	0	

Load combination: 1.0G + 1.0ψ_0Q + 1.0S + 0.5W (Service)

Node	Deflection		Rotation	Co-ordinate system
	X	Z		
	(mm)	(mm)	(°)	
1	0	0	0	
2	0	0.1	0.06054	
3	0	0.1	-0.06032	
4	0	0	0	

Load combination: 1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X	Z		
	(mm)	(mm)	(°)	
1	0	0	0	
2	0	0.1	0.08175	
3	0	0.1	-0.0814	
4	0	0	0	

Load combination: 1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service)

Node	Deflection		Rotation	Co-ordinate system
	X	Z		
	(mm)	(mm)	(°)	
1	0	0	0	
2	0	0.1	0.06055	
3	0	0.1	-0.06031	
4	0	0	0	

		Project			Job Ref.	
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Load combination: 1.0G + 1.5W (Strength)

Node	Deflection		Rotation	Co-ordinate system
	X	**Z**		
	(mm)	**(mm)**	**(°)**	
1	0	0	0	
2	0	0.1	0.06056	
3	0	0.1	-0.0603	
4	0	0	0	

Load combination: 1.0G + 1.0W (Service)

Node	Deflection		Rotation	Co-ordinate system
	X	**Z**		
	(mm)	**(mm)**	**(°)**	
1	0	0	0	
2	0	0.1	0.06053	
3	0	0.1	-0.06033	
4	0	0	0	

Reactions

Load case: Self Weight

Node	Force		Moment
	Fx	**Fz**	**My**
	(kN)	**(kN)**	**(kNm)**
1	1	3.4	1
4	-1	3.4	-1

Load case: Permanent

Node	Force		Moment
	Fx	**Fz**	**My**
	(kN)	**(kN)**	**(kNm)**
1	1.4	5.3	1.4
4	-1.4	7.2	-1.4

Load case: Snow

Node	Force		Moment
	Fx	**Fz**	**My**
	(kN)	**(kN)**	**(kNm)**
1	0	1.7	0
4	0	3.4	0

Load case: Wind

Node	Force		Moment
	Fx	**Fz**	**My**
	(kN)	**(kN)**	**(kNm)**
1	0	15.8	0
4	0	21	0

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Load combination: $1.35G + 1.5\psi_0Q + 1.5S$ (Strength)

Node	Force		Moment
	Fx	**Fz**	**My**
	(kN)	**(kN)**	**(kNm)**
1	3.3	14.2	3.2
4	-3.3	19.3	-3.2

Load combination: $1.35G + 1.5Q + 1.5\psi_0S + 1.5\psi_0W$ (Strength)

Node	Force		Moment
	Fx	**Fz**	**My**
	(kN)	**(kN)**	**(kNm)**
1	3.3	24.7	3.2
4	-3.3	32.5	-3.2

Load combination: $1.0G + 1.0\psi_0Q + 1.0S + 0.5W$ (Service)

Node	Force		Moment
	Fx	**Fz**	**My**
	(kN)	**(kN)**	**(kNm)**
1	2.4	18.2	2.3
4	-2.4	24.4	-2.4

Load combination: $1.35G + 1.5\psi_0Q + 1.5\psi_0S + 1.5W$ (Strength)

Node	Force		Moment
	Fx	**Fz**	**My**
	(kN)	**(kN)**	**(kNm)**
1	3.3	36.5	3.2
4	-3.3	48.2	-3.2

Load combination: $1.0G + 1.0\psi_0Q + 0.5S + 1.0W$ (Service)

Node	Force		Moment
	Fx	**Fz**	**My**
	(kN)	**(kN)**	**(kNm)**
1	2.4	25.2	2.3
4	-2.4	33.2	-2.4

Load combination: $1.0G + 1.5W$ (Strength)

Node	Force		Moment
	Fx	**Fz**	**My**
	(kN)	**(kN)**	**(kNm)**
1	2.4	32.2	2.3
4	-2.4	42	-2.4

Load combination: $1.0G + 1.0W$ (Service)

Node	Force		Moment
	Fx	**Fz**	**My**
	(kN)	**(kN)**	**(kNm)**
1	2.4	24.4	2.3
4	-2.4	31.5	-2.4

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Element end forces

Load combination: 1.35G + 1.5ψ_0Q + 1.5S (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-14.2	3.3	-3.2
		2	13	-3.3	-6.7
2	8.4	2	-3.3	-7.9	6.7
		3	3.3	-7.9	-6.7
3	3	3	-18.1	-3.3	6.7
		4	19.3	3.3	3.2

Load combination: 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-24.7	3.3	-3.2
		2	11.7	-3.3	-6.7
2	8.4	2	-3.3	-7.9	6.7
		3	3.3	-7.9	-6.7
3	3	3	-15.5	-3.3	6.7
		4	32.5	3.3	3.2

Load combination: 1.0G + 1.0ψ_0Q + 1.0S + 0.5W (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-18.2	2.4	-2.3
		2	9.4	-2.4	-4.9
2	8.4	2	-2.4	-5.8	4.9
		3	2.4	-5.8	-4.9
3	3	3	-13	-2.4	4.9
		4	24.4	2.4	2.4

Load combination: 1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-36.5	3.3	-3.2
		2	11.7	-3.3	-6.7
2	8.4	2	-3.3	-7.9	6.7
		3	3.3	-7.9	-6.7
3	3	3	-15.5	-3.3	6.7
		4	48.2	3.3	3.2

Load combination: 1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-25.2	2.4	-2.3
		2	8.6	-2.4	-4.9
2	8.4	2	-2.4	-5.8	4.9
		3	2.4	-5.8	-4.9
3	3	3	-11.3	-2.4	4.9
		4	33.2	2.4	2.4

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Load combination: 1.0G + 1.5W (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-32.2	2.4	-2.3
		2	7.7	-2.4	-4.9
2	8.4	2	-2.4	-5.8	4.9
		3	2.4	-5.8	-4.9
3	3	3	-9.6	-2.4	4.9
		4	42	2.4	2.4

Load combination: 1.0G + 1.0W (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	3	1	-24.4	2.4	-2.3
		2	7.7	-2.4	-4.9
2	8.4	2	-2.4	-5.8	4.9
		3	2.4	-5.8	-4.9
3	3	3	-9.6	-2.4	4.9
		4	31.5	2.4	2.4

Forces

Member results

Load combination: 1.0G + 1.0ψ_0Q + 1.0S + 0.5W (Service)

Member	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
Column 1	0	-2.4	0	2.3	3	-4.9 (min)
Beam 1	0	5.8 (max abs)	4.2	7.3 (max)	0	-4.9 (min)
Column 2	0	2.4	3	2.4	0	-4.9

Load combination: 1.0G + 1.0ψ_0Q + 1.0S + 0.5W (Service)

Member	Deflection			
	Pos (m)	Max (mm)	Pos (m)	Min (mm)
Column 1	3	0	1.979	-0.5
Beam 1	4.204	3.8 (max)	0	0.1
Column 2	0	0	1.017	-0.5 (min)

Member results

Load combination: 1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service)

Member	Shear force		Moment			
	Pos (m)	Max abs (kN)	Pos (m)	Max (kNm)	Pos (m)	Min (kNm)
Column 1	0	-2.4	0	2.3	3	-4.9 (min)
Beam 1	0	5.8 (max abs)	4.2	7.3 (max)	0	-4.9 (min)
Column 2	0	2.4	3	2.4	0	-4.9

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Load combination: 1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service)

Member	Deflection			
	Pos	Max	Pos	Min
	(m)	(mm)	(m)	(mm)
Column 1	3	0	1.979	-0.5
Beam 1	4.204	3.8 (max)	0	0.1
Column 2	0	0	1.017	-0.5 (min)

Member results

Load combination: 1.0G + 1.0W (Service)

Member	Shear force		Moment			
	Pos	Max abs	Pos	Max	Pos	Min
	(m)	(kN)	(m)	(kNm)	(m)	(kNm)
Column 1	0	-2.4	0	2.3	3	-4.9 (min)
Beam 1	0	5.8 (max abs)	4.2	7.3 (max)	0	-4.9 (min)
Column 2	0	2.4	3	2.4	0	-4.9

Load combination: 1.0G + 1.0W (Service)

Member	Deflection			
	Pos	Max	Pos	Min
	(m)	(mm)	(m)	(mm)
Column 1	3	0	1.98	-0.5
Beam 1	4.204	3.8 (max)	0	0.1
Column 2	0	0	1.017	-0.5 (min)

;

Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = **1**

Resistance of members to instability; γ_{M1} = **1**

Resistance of tensile members to fracture; γ_{M2} = **1.1**

Column 1 design

Section details

Section type;   UC 152x152x30 (BS4-1)

Steel grade;   User defined

Nominal thickness of element; t_{nom} = max(t_f, t_w) = **9.4** mm

Nominal yield strength; f_y = **355** N/mm^2

Nominal ultimate tensile strength; f_u = **470** N/mm^2

Modulus of elasticity;   E = **210000** N/mm^2

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UC 152x152x30 (BS4-1)
Section depth, h, 157.6 mm
Section breadth, b, 152.9 mm
Mass of section, Mass, 30 kg/m
Flange thickness, t_f, 9.4 mm
Web thickness, t_w, 6.5 mm
Root radius, r, 7.6 mm
Area of section, A, 3826 mm²
Radius of gyration about y-axis, i_y, 67.591 mm
Radius of gyration about z-axis, i_z, 38.272 mm
Elastic section modulus about y-axis, $W_{el.y}$, 221833 mm³
Elastic section modulus about z-axis, $W_{el.z}$, 73311 mm³
Plastic section modulus about y-axis, $W_{pl.y}$, 247665 mm³
Plastic section modulus about z-axis, $W_{pl.z}$, 111590 mm³
Second moment of area about y-axis, I_y, 17480413 mm⁴
Second moment of area about z-axis, I_z, 5604633 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5ψ_0Q + 1.5S (Strength)

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = \textbf{0.81}$

Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)

Width of section; $c = d = \textbf{123.6}$ mm

$\alpha = min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) = \textbf{0.524}$

$c / t_w = 19 = 23.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1);$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = \textbf{65.6}$ mm

$c / t_f = 7 = 8.6 \times \varepsilon <= 9 \times \varepsilon;$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force; $N_{Ed} = \textbf{13.9}$ kN

Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = \textbf{1358.3}$ kN

$N_{Ed} / N_{c,Rd} = \textbf{0.01}$

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,y} = L_{m1\_s1} = \textbf{3000}$ mm

Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = \textbf{4025.6}$ kN

Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} = \textbf{0.581}$

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2;   b

Imperfection factor - Table 6.1; $\alpha_y = \textbf{0.34}$

Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) = \textbf{0.733}$

Buckling reduction factor - eq 6.49; $\chi_y = min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1) = \textbf{0.847}$

Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = \textbf{1149.9}$ kN

$N_{Ed} / N_{b,y,Rd} = \textbf{0.012}$

PASS - Design buckling resistance exceeds design compression

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Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,z} = L_{m1\_s1\_seg1} = \textbf{3000}$ mm

Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 = \textbf{1290.7}$ kN

Slenderness ratio for buckling - eq 6.50; $\overline{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} = \textbf{1.026}$

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; c

Imperfection factor - Table 6.1; $\alpha_z = \textbf{0.49}$

Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\overline{\lambda}_z - 0.2) + \overline{\lambda}_z^2) = \textbf{1.229}$

Buckling reduction factor - eq 6.49; $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \overline{\lambda}_z^2)}), 1) = \textbf{0.525}$

Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = \textbf{713.2}$ kN

 $N_{Ed} / N_{b,z,Rd} = \textbf{0.019}$

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length; $L_{cr,T} = L_{m1\_s1\_seg1\_R} = \textbf{3000}$ mm

Distance from shear centre to centroid in y axis; $y_0 = \textbf{0.0}$ mm

Distance from shear centre to centroid in z axis; $z_0 = \textbf{0.0}$ mm

Radius of gyration; $i_0 = \sqrt{(i_y^2 + i_z^2)} = \textbf{77.7}$ mm

Elastic critical torsional buckling force; $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) = \textbf{2582.7}$ kN

Torsion factor; $\beta_T = 1 - (y_0 / i_0)^2 = \textbf{1}$

Elastic critical torsional-flexural buckling force

 $N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}] = \textbf{2582.7}$ kN

Elastic critical buckling force; $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) = \textbf{2582.7}$ kN

Slenderness ratio for torsional buckling - eq 6.52; $\overline{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]} = \textbf{0.725}$

Design resistance for torsional and torsional-flexural buckling - Section 6.3.1.1

Buckling curve - Table 6.2; c

Imperfection factor - Table 6.1; $\alpha_T = \textbf{0.49}$

Buckling reduction determination factor; $\phi_T = 0.5 \times (1 + \alpha_T \times (\overline{\lambda}_T - 0.2) + \overline{\lambda}_T^2) = \textbf{0.892}$

Buckling reduction factor - eq 6.49; $\chi_T = \min(1 / (\phi_T + \sqrt{(\phi_T^2 - \overline{\lambda}_T^2)}), 1) = \textbf{0.709}$

Design buckling resistance - eq 6.47; $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} = \textbf{963.1}$ kN

 $N_{Ed} / N_{b,T,Rd} = \textbf{0.014}$

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = \textbf{138.8}$ mm; $\eta = \textbf{1.000}$

 $h_w / t_w = 21.4 = 26.2 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{y,Ed} = \textbf{3.6}$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \textbf{1156}$ mm^2

Design shear resistance - cl 6.2.6(2); $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} = \textbf{236.9}$ kN

 $V_{y,Ed} / V_{c,y,Rd} = \textbf{0.015}$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} = \textbf{4}$ kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \textbf{87.9}$ kNm

 $M_{y,Ed} / M_{c,y,Rd} = \textbf{0.045}$

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PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - For cantilever beams; $k_c = 1$

$C_1 = 1 / k_c^2 = 1$

Poissons ratio; $\nu = 0.3$

Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769$ N/mm^2

Unrestrained effective length; $L = 1.0 \times L_{m1\_s1\_seg1\_T} = 3000$ mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 141.8$ kNm

Slenderness ratio for lateral torsional buckling; $\overline{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 0.787$

Limiting slenderness ratio; $\overline{\lambda}_{LT,0} = 0.4$

$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0}$ *- Lateral torsional buckling cannot be ignored*

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$

Correction factor for rolled sections; $\beta = 0.75$

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta \times \overline{\lambda}_{LT}^2] = 0.798$

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \overline{\lambda}_{LT}^2)}], 1, 1 / \overline{\lambda}_{LT}^2) = 0.824$

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\overline{\lambda}_{LT} - 0.8)^2], 1) = 1.000$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \overline{\lambda}_{LT}^2) = 0.824$

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = 72.5$ kNm

$M_{y,Ed} / M_{b,y,Rd} = 0.055$

PASS - Design buckling resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Bending and axial force check - eq.6.33 & eq.6.34; $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0}) = 160.1$ kN

$N_{Ed} / N_{y,lim} = 0.087$

Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; $\psi_y = 3.957$ kNm / -6.742 kNm = **-0.587**

$\alpha_y = -1.392$ kNm / -6.742 kNm = **0.206**

$C_{my} = \max(0.6 + 0.4 \times \psi_y) = 0.365$

; $\psi_{LT} = 3.957$ kNm / -6.742 kNm = **-0.587**

$\alpha_{LT} = -1.392$ kNm / -6.742 kNm = **0.206**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}) = 0.365$

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y = 87.9$ kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y = 39.6$ kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y = 1358.3$ kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\overline{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = 0.367$

$k_{zy} = 1 - 0.1 \times \min(1, \overline{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) = 0.983$

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = 0.032$

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = 0.073$

PASS - Combined bending and compression checks are satisfied

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Check design at end of span

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = $ **138.8** mm; $\eta = $ **1.000**

$h_w / t_w = 21.4 = 26.2 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{y,Ed} = $ **3.6** kN

Shear area - cl 6.2.6(3); $A_v = max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = $ **1156** mm²

Design shear resistance - cl 6.2.6(2); $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = $ **236.9** kN

$V_{y,Ed} / V_{c,y,Rd} = $ **0.015**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} = $ **6.7** kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = $ **87.9** kNm

$M_{y,Ed} / M_{c,y,Rd} = $ **0.077**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - For cantilever beams; $k_c = $ **1**

$C_1 = 1 / k_c^2 = $ **1**

Poissons ratio; $\nu = $ **0.3**

Shear modulus; $G = E / [2 \times (1 + \nu)] = $ **80769** N/mm²

Unrestrained effective length; $L = 1.0 \times L_{m1_s1_seg1_B} = $ **3000** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = $ **141.8** kNm

Slenderness ratio for lateral torsional buckling; $\overline{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = $ **0.787**

Limiting slenderness ratio; $\overline{\lambda}_{LT,0} = $ **0.4**

$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0}$ *- Lateral torsional buckling cannot be ignored*

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5;   b

Imperfection factor - Table 6.3; $\alpha_{LT} = $ **0.34**

Correction factor for rolled sections; $\beta = $ **0.75**

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta \times \overline{\lambda}_{LT}^2] = $ **0.798**

LTB reduction factor - eq 6.57; $\chi_{LT} = min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \overline{\lambda}_{LT}^2)}], 1, 1 / \overline{\lambda}_{LT}^2) = $ **0.824**

Modification factor; $f = min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\overline{\lambda}_{LT} - 0.8)^2], 1) = $ **1.000**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = min(\chi_{LT} / f, 1, 1 / \overline{\lambda}_{LT}^2) = $ **0.824**

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = $ **72.5** kNm

$M_{y,Ed} / M_{b,y,Rd} = $ **0.093**

PASS - Design buckling resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; $\psi_y = $ 3.957 kNm / -6.742 kNm = **-0.587**

$\alpha_y = $ -1.392 kNm / -6.742 kNm = **0.206**

$C_{my} = max(0.6 + 0.4 \times \psi_y) = $ **0.365**

; $\psi_{LT} = $ 3.957 kNm / -6.742 kNm = **-0.587**

$\alpha_{LT} = $ -1.392 kNm / -6.742 kNm = **0.206**

$C_{mLT} = max(0.6 + 0.4 \times \psi_{LT}) = $ **0.365**

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Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y = $ **87.9** kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y = $ **39.6** kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y = $ **1358.3** kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = $ **0.367**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) = $ **0.985**

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = $ **0.045**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = $ **0.109**

PASS - Combined bending and compression checks are satisfied

Consider Combination 7 - 1.0G + 1.0W (Service)

Check design 2259 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection; $\delta_y = $ **0.7** mm

Allowable deflection; $\delta_{y,Allowable} = L_{m1_s1} / 180 = $ **16.7** mm

$\delta_y / \delta_{y,Allowable} = $ **0.044**

PASS - Allowable deflection exceeds design deflection

Beam 1 design

Section details

Section type;   UC 203x203x60 (BS4-1)

Steel grade - EN 10025-2:2004;   S355

Nominal thickness of element; $t_{nom} = \max(t_f, t_w) = $ **14.2** mm

Nominal yield strength; $f_y = $ **355** N/mm^2

Nominal ultimate tensile strength; $f_u = $ **470** N/mm^2

Modulus of elasticity; $E = $ **210000** N/mm^2
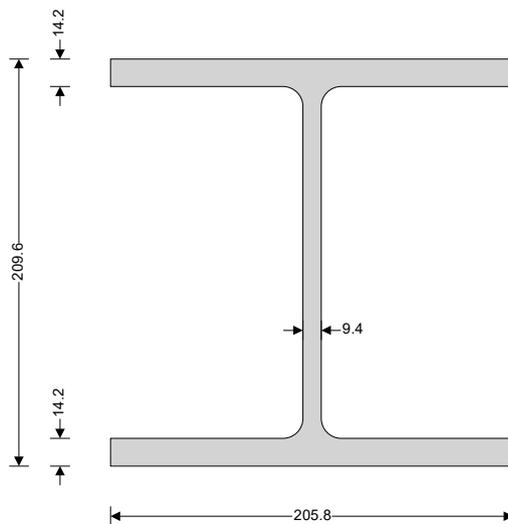
UC 203x203x60 (BS4-1)
Section depth, h, 209.6 mm
Section breadth, b, 205.8 mm
Mass of section, Mass, 60 kg/m
Flange thickness, t_f, 14.2 mm
Web thickness, t_w, 9.4 mm
Root radius, r, 10.2 mm
Area of section, A, 7637 mm^2
Radius of gyration about y-axis, i_y, 89.55 mm
Radius of gyration about z-axis, i_z, 51.993 mm
Elastic section modulus about y-axis, $W_{el.y}$, 584400 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 200641 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 656076 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 305336 mm^3
Second moment of area about y-axis, I_y, 61245135 mm^4
Second moment of area about z-axis, I_z, 20645981 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

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Consider Combination 4 - 1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)

Classification of cross sections - Section 5.5

$$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = \mathbf{0.81}$$

Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)

Width of section; $c = d = \mathbf{160.8}$ mm

$\alpha = min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) = \mathbf{0.503}$

$c / t_w = 17.1 = 21 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1);$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = \mathbf{88}$ mm

$c / t_f = 6.2 = 7.6 \times \varepsilon <= 9 \times \varepsilon;$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force; $N_{Ed} = \mathbf{3.6}$ kN

Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = \mathbf{2711.2}$ kN

$N_{Ed} / N_{c,Rd} = \mathbf{0.001}$

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,y} = L_{m2\_s1} = \mathbf{8400}$ mm

Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = \mathbf{1799}$ kN

Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} = \mathbf{1.228}$

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2;   b

Imperfection factor - Table 6.1; $\alpha_y = \mathbf{0.34}$

Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) = \mathbf{1.428}$

Buckling reduction factor - eq 6.49; $\chi_y = min(1 / (\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}), 1) = \mathbf{0.463}$

Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = \mathbf{1256.3}$ kN

$N_{Ed} / N_{b,y,Rd} = \mathbf{0.003}$

PASS - Design buckling resistance exceeds design compression

Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,z} = L_{m2\_s1\_seg1} = \mathbf{8400}$ mm

Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 = \mathbf{606.5}$ kN

Slenderness ratio for buckling - eq 6.50; $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} = \mathbf{2.114}$

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2;   c

Imperfection factor - Table 6.1; $\alpha_z = \mathbf{0.49}$

Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) = \mathbf{3.204}$

Buckling reduction factor - eq 6.49; $\chi_z = min(1 / (\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}), 1) = \mathbf{0.178}$

Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = \mathbf{483.1}$ kN

$N_{Ed} / N_{b,z,Rd} = \mathbf{0.007}$

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length; $L_{cr,T} = L_{m2\_s1\_seg1\_R} = \mathbf{8400}$ mm

Distance from shear centre to centroid in y axis; $y_0 = \mathbf{0.0}$ mm

Distance from shear centre to centroid in z axis; $z_0 = \mathbf{0.0}$ mm

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Radius of gyration; $i_0 = \sqrt{(i_y^2 + i_z^2)} = $ **103.5** mm

Elastic critical torsional buckling force; $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) = $ **4097.6** kN

Torsion factor; $\beta_T = 1 - (y_0 / i_0)^2 = $ **1**

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}] = $ **1799** kN

Elastic critical buckling force; $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) = $ **1799** kN

Slenderness ratio for torsional buckling - eq 6.52; $\overline{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]} = $ **1.228**

Design resistance for torsional and torsional-flexural buckling - Section 6.3.1.1

Buckling curve - Table 6.2; c

Imperfection factor - Table 6.1; $\alpha_T = $ **0.49**

Buckling reduction determination factor; $\phi_T = 0.5 \times (1 + \alpha_T \times (\overline{\lambda}_T - 0.2) + \overline{\lambda}_T^2) = $ **1.505**

Buckling reduction factor - eq 6.49; $\chi_T = \min(1 / (\phi_T + \sqrt{(\phi_T^2 - \overline{\lambda}_T^2)}), 1) = $ **0.421**

Design buckling resistance - eq 6.47; $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} = $ **1140.9** kN

$N_{Ed} / N_{b,T,Rd} = $ **0.003**

PASS - Design buckling resistance exceeds design compression

Check design 4049 mm along span

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} = $ **8.6** kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = $ **232.9** kNm

$M_{y,Ed} / M_{c,y,Rd} = $ **0.037**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $k_c = $ **0.845**

$C_1 = 1 / k_c^2 = $ **1.4**

Poissons ratio; $\nu = $ **0.3**

Shear modulus; $G = E / [2 \times (1 + \nu)] = $ **80769** N/mm^2

Unrestrained effective length; $L = 1.0 \times L_{m2_s1_seg1_T} = $ **8400** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = $ **228.6** kNm

Slenderness ratio for lateral torsional buckling; $\overline{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = $ **1.009**

Limiting slenderness ratio; $\overline{\lambda}_{LT,0} = $ **0.4**

$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0}$ - *Lateral torsional buckling cannot be ignored*

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} = $ **0.34**

Correction factor for rolled sections; $\beta = $ **0.75**

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\overline{\lambda}_{LT} - \overline{\lambda}_{LT,0}) + \beta \times \overline{\lambda}_{LT}^2] = $ **0.986**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \overline{\lambda}_{LT}^2)}], 1, 1 / \overline{\lambda}_{LT}^2) = $ **0.694**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\overline{\lambda}_{LT} - 0.8)^2], 1) = $ **0.929**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \overline{\lambda}_{LT}^2) = $ **0.747**

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = $ **173.9** kNm

$M_{y,Ed} / M_{b,y,Rd} = $ **0.05**

PASS - Design buckling resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Bending and axial force check - eq.6.33 & eq.6.34; $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0}) = $ **231.6** kN

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$$N_{Ed} / N_{y,lim} = \textbf{0.015}$$

Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; $\quad \psi_y$ = -6.739 kNm / -9.121 kNm = **0.739**

α_y = 8.596 kNm / -9.121 kNm = **-0.942**

C_{my} = max(0.2 + 0.8 × α_y, 0.4) = **0.400**

; $\quad \psi_{LT}$ = -6.739 kNm / -9.121 kNm = **0.739**

α_{LT} = 8.596 kNm / -9.121 kNm = **-0.942**

C_{mLT} = max(0.2 + 0.8 × α_{LT}, 0.4) = **0.400**

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $\quad M_{y,Rk} = W_{pl.y} \times f_y$ = **232.9 kNm**

Characteristic moment resistance; $\quad M_{z,Rk} = W_{pl.z} \times f_y$ = **108.4 kNm**

Characteristic resistance to normal force; $\quad N_{Rk} = A \times f_y$ = **2711.2 kN**

Interaction factors; $\quad k_{yy} = C_{my} \times (1 + min(\ \overline{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.401**

$k_{zy} = 1 - 0.1 \times min(1,\ \overline{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.995**

Interaction formulae - eq 6.61 & eq 6.62; $\quad N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.024**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.06**

PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web; $\quad h_w = h - 2 \times t_f$ = **181.2 mm**; $\quad \eta$ = **1.000**

$h_w / t_w = 19.3 = 23.7 \times \epsilon / \eta < 72 \times \epsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $\quad V_{y,Ed}$ = **8.2 kN**

Shear area - cl 6.2.6(3); $\quad A_v = max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **2216 mm²**

Design shear resistance - cl 6.2.6(2); $\quad V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3})) / \gamma_{M0}$ = **454.1 kN**

$V_{y,Ed} / V_{c,y,Rd}$ = **0.018**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; $\quad M_{y,Ed}$ = **9.1 kNm**

Design bending resistance moment - eq 6.13; $\quad M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **232.9 kNm**

$M_{y,Ed} / M_{c,y,Rd}$ = **0.039**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; $\quad k_c$ = **0.845**

$C_1 = 1 / k_c^2$ = **1.4**

Poissons ratio; $\quad \nu$ = **0.3**

Shear modulus; $\quad G = E / [2 \times (1 + \nu)]$ = **80769 N/mm²**

Unrestrained effective length; $\quad L = 1.0 \times L_{m2\_s1\_seg1\_B}$ = **8400 mm**

Elastic critical buckling moment; $\quad M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **228.6 kNm**

Slenderness ratio for lateral torsional buckling; $\quad \overline{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.009**

Limiting slenderness ratio; $\quad \overline{\lambda}_{LT,0}$ = **0.4**

$\overline{\lambda}_{LT}$ > $\overline{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

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Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5;   b

Imperfection factor - Table 6.3; $\alpha_{LT} = $ **0.34**

Correction factor for rolled sections; $\beta = $ **0.75**

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = $ **0.986**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = $ **0.694**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = $ **0.929**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = $ **0.747**

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = $ **173.9** kNm

$M_{y,Ed} / M_{b,y,Rd} = $ **0.052**

PASS - Design buckling resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; $\psi_y = $ -6.739 kNm / -9.121 kNm = **0.739**

$\alpha_y = $ 8.596 kNm / -9.121 kNm = **-0.942**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4) = $ **0.400**

; $\psi_{LT} = $ -6.739 kNm / -9.121 kNm = **0.739**

$\alpha_{LT} = $ 8.596 kNm / -9.121 kNm = **-0.942**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4) = $ **0.400**

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y = $ **232.9** kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y = $ **108.4** kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y = $ **2711.2** kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = $ **0.401**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) = $ **0.995**

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = $ **0.025**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = $ **0.064**

PASS - Combined bending and compression checks are satisfied

Consider Combination 5 - 1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service)

Check design 4100 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection; $\delta_y = $ **3.1** mm

Allowable deflection; $\delta_{y,Allowable} = L_{m2_s1} / 360 = $ **23.3** mm

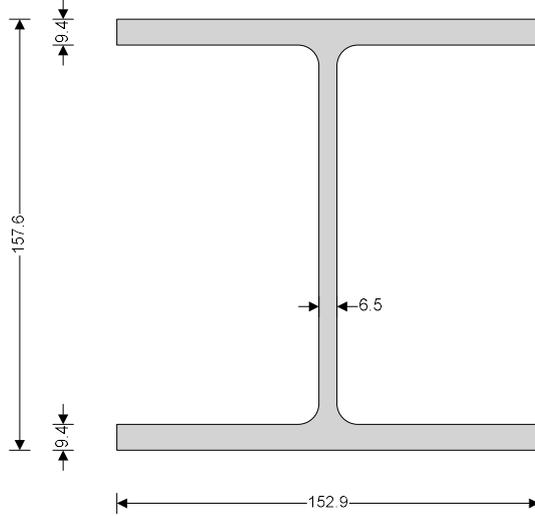$\delta_y / \delta_{y,Allowable} = $ **0.133**

PASS - Allowable deflection exceeds design deflection

Column 2 design

Section details

Section type;   UC 152x152x30 (BS4-1)

Steel grade - EN 10025-2:2004;   S355

Nominal thickness of element; $t_{nom} = \max(t_f, t_w) = $ **9.4** mm

Nominal yield strength; $f_y = $ **355** N/mm^2

Nominal ultimate tensile strength; $f_u = $ **470** N/mm^2

Modulus of elasticity; $E = $ **210000** N/mm^2

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UC 152x152x30 (BS4-1)
Section depth, h, 157.6 mm
Section breadth, b, 152.9 mm
Mass of section, Mass, 30 kg/m
Flange thickness, t_f, 9.4 mm
Web thickness, t_w, 6.5 mm
Root radius, r, 7.6 mm
Area of section, A, 3826 mm²
Radius of gyration about y-axis, i_y, 67.591 mm
Radius of gyration about z-axis, i_z, 38.272 mm
Elastic section modulus about y-axis, $W_{el.y}$, 221833 mm³
Elastic section modulus about z-axis, $W_{el.z}$, 73311 mm³
Plastic section modulus about y-axis, $W_{pl.y}$, 247665 mm³
Plastic section modulus about z-axis, $W_{pl.z}$, 111590 mm³
Second moment of area about y-axis, I_y, 17480413 mm⁴
Second moment of area about z-axis, I_z, 5604633 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5ψ_0Q + 1.5S (Strength)

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \textbf{0.81}$

Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)

Width of section; $c = d = \textbf{123.6}$ mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) = \textbf{0.534}$

$c / t_w = 19 = 23.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1);$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2 = \textbf{65.6}$ mm

$c / t_f = 7 = 8.6 \times \varepsilon <= 9 \times \varepsilon;$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force; $N_{Ed} = \textbf{18.1}$ kN

Design resistance of section - eq 6.10; $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = \textbf{1358.3}$ kN

$N_{Ed} / N_{c,Rd} = \textbf{0.013}$

PASS - Design compression resistance exceeds design compression

Slenderness ratio for y-y axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,y} = L_{m3\_s1} = \textbf{3000}$ mm

Critical buckling force; $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = \textbf{4025.6}$ kN

Slenderness ratio for buckling - eq 6.50; $\overline{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} = \textbf{0.581}$

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2;   b

Imperfection factor - Table 6.1; $\alpha_y = \textbf{0.34}$

Buckling reduction determination factor; $\phi_y = 0.5 \times (1 + \alpha_y \times (\overline{\lambda}_y - 0.2) + \overline{\lambda}_y^2) = \textbf{0.733}$

Buckling reduction factor - eq 6.49; $\chi_y = \min(1 / (\phi_y + \sqrt{(\phi_y^2 - \overline{\lambda}_y^2)}), 1) = \textbf{0.847}$

Design buckling resistance - eq 6.47; $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = \textbf{1149.9}$ kN

$N_{Ed} / N_{b,y,Rd} = \textbf{0.016}$

PASS - Design buckling resistance exceeds design compression

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Slenderness ratio for z-z axis flexural buckling - Section 6.3.1.3

Critical buckling length; $L_{cr,z} = L_{m3\_s1\_seg1} = \textbf{3000}$ mm

Critical buckling force; $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 = \textbf{1290.7}$ kN

Slenderness ratio for buckling - eq 6.50; $\overline{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} = \textbf{1.026}$

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2; c

Imperfection factor - Table 6.1; $\alpha_z = \textbf{0.49}$

Buckling reduction determination factor; $\phi_z = 0.5 \times (1 + \alpha_z \times (\overline{\lambda}_z - 0.2) + \overline{\lambda}_z^2) = \textbf{1.229}$

Buckling reduction factor - eq 6.49; $\chi_z = \min(1 / (\phi_z + \sqrt{(\phi_z^2 - \overline{\lambda}_z^2)}), 1) = \textbf{0.525}$

Design buckling resistance - eq 6.47; $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = \textbf{713.2}$ kN

$N_{Ed} / N_{b,z,Rd} = \textbf{0.025}$

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length; $L_{cr,T} = L_{m3\_s1\_seg1\_R} = \textbf{3000}$ mm

Distance from shear centre to centroid in y axis; $y_0 = \textbf{0.0}$ mm

Distance from shear centre to centroid in z axis; $z_0 = \textbf{0.0}$ mm

Radius of gyration; $i_0 = \sqrt{(i_y^2 + i_z^2)} = \textbf{77.7}$ mm

Elastic critical torsional buckling force; $N_{cr,T} = 1 / i_0^2 \times (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) = \textbf{2582.7}$ kN

Torsion factor; $\beta_T = 1 - (y_0 / i_0)^2 = \textbf{1}$

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}] = \textbf{2582.7}$ kN

Elastic critical buckling force; $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) = \textbf{2582.7}$ kN

Slenderness ratio for torsional buckling - eq 6.52; $\overline{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]} = \textbf{0.725}$

Design resistance for torsional and torsional-flexural buckling - Section 6.3.1.1

Buckling curve - Table 6.2; c

Imperfection factor - Table 6.1; $\alpha_T = \textbf{0.49}$

Buckling reduction determination factor; $\phi_T = 0.5 \times (1 + \alpha_T \times (\overline{\lambda}_T - 0.2) + \overline{\lambda}_T^2) = \textbf{0.892}$

Buckling reduction factor - eq 6.49; $\chi_T = \min(1 / (\phi_T + \sqrt{(\phi_T^2 - \overline{\lambda}_T^2)}), 1) = \textbf{0.709}$

Design buckling resistance - eq 6.47; $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} = \textbf{963.1}$ kN

$N_{Ed} / N_{b,T,Rd} = \textbf{0.019}$

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = \textbf{138.8}$ mm; $\eta = \textbf{1.000}$

$h_w / t_w = 21.4 = 26.2 \times \epsilon / \eta < 72 \times \epsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{y,Ed} = \textbf{3.3}$ kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \textbf{1156}$ mm^2

Design shear resistance - cl 6.2.6(2); $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} = \textbf{236.9}$ kN

$V_{y,Ed} / V_{c,y,Rd} = \textbf{0.014}$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} = \textbf{6.7}$ kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \textbf{87.9}$ kNm

$M_{y,Ed} / M_{c,y,Rd} = \textbf{0.076}$

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PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - For cantilever beams; $k_c = 1$

$C_1 = 1 / k_c^2 = 1$

Poissons ratio; $\nu = 0.3$

Shear modulus; $G = E / [2 \times (1 + \nu)] = 80769$ N/mm²

Unrestrained effective length; $L = 1.0 \times L_{m3\_s1\_seg1\_B} = 3000$ mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 141.8$ kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = 0.787$

Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = 0.4$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ *- Lateral torsional buckling cannot be ignored*

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5; b

Imperfection factor - Table 6.3; $\alpha_{LT} = 0.34$

Correction factor for rolled sections; $\beta = 0.75$

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.798$

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.824$

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 1.000$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = 0.824$

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = 72.5$ kNm

$M_{y,Ed} / M_{b,y,Rd} = 0.092$

PASS - Design buckling resistance moment exceeds design bending moment

Check bending and axial force - Section 6.2.9

Bending and axial force check - eq.6.33 & eq.6.34; $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0}) = 160.1$ kN

$N_{Ed} / N_{y,lim} = 0.113$

Allowance need not be made for the effect of the axial force on the plastic resistance moment about the y-y axis

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; $\psi_y = 3.175$ kNm / -6.655 kNm = **-0.477**

$\alpha_y = -1.74$ kNm / -6.655 kNm = **0.261**

$C_{my} = \max(0.6 + 0.4 \times \psi_y) = 0.409$

; $\psi_{LT} = 3.175$ kNm / -6.655 kNm = **-0.477**

$\alpha_{LT} = -1.74$ kNm / -6.655 kNm = **0.261**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}) = 0.409$

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y = 87.9$ kNm

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y = 39.6$ kNm

Characteristic resistance to normal force; $N_{Rk} = A \times f_y = 1358.3$ kN

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = 0.412$

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) = 0.984$

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = 0.054$

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = 0.116$

PASS - Combined bending and compression checks are satisfied

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Check design at end of span

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f = \textbf{138.8 mm}$; $\eta = \textbf{1.000}$

$h_w / t_w = 21.4 = 26.2 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force; $V_{y,Ed} = \textbf{3.3 kN}$

Shear area - cl 6.2.6(3); $A_v = max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \textbf{1156 mm}^2$

Design shear resistance - cl 6.2.6(2); $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} = \textbf{236.9 kN}$

$V_{y,Ed} / V_{c,y,Rd} = \textbf{0.014}$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed} = \textbf{3.2 kNm}$

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \textbf{87.9 kNm}$

$M_{y,Ed} / M_{c,y,Rd} = \textbf{0.036}$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - For cantilever beams; $k_c = \textbf{1}$

$C_1 = 1 / k_c^2 = \textbf{1}$

Poissons ratio; $\nu = \textbf{0.3}$

Shear modulus; $G = E / [2 \times (1 + \nu)] = \textbf{80769 N/mm}^2$

Unrestrained effective length; $L = 1.0 \times L_{m3\_s1\_seg1\_T} = \textbf{3000 mm}$

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = \textbf{141.8 kNm}$

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \textbf{0.787}$

Limiting slenderness ratio; $\bar{\lambda}_{LT,0} = \textbf{0.4}$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ ***- Lateral torsional buckling cannot be ignored***

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5;   b

Imperfection factor - Table 6.3; $\alpha_{LT} = \textbf{0.34}$

Correction factor for rolled sections; $\beta = \textbf{0.75}$

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \textbf{0.798}$

LTB reduction factor - eq 6.57; $\chi_{LT} = min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \textbf{0.824}$

Modification factor; $f = min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \textbf{1.000}$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = \textbf{0.824}$

Design buckling resistance moment - eq 6.55; $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \textbf{72.5 kNm}$

$M_{y,Ed} / M_{b,y,Rd} = \textbf{0.044}$

PASS - Design buckling resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3; $\psi_y = 3.175 \text{ kNm} / -6.655 \text{ kNm} = \textbf{-0.477}$

$\alpha_y = -1.74 \text{ kNm} / -6.655 \text{ kNm} = \textbf{0.261}$

$C_{my} = max(0.6 + 0.4 \times \psi_y) = \textbf{0.409}$

; $\psi_{LT} = 3.175 \text{ kNm} / -6.655 \text{ kNm} = \textbf{-0.477}$

$\alpha_{LT} = -1.74 \text{ kNm} / -6.655 \text{ kNm} = \textbf{0.261}$

$C_{mLT} = max(0.6 + 0.4 \times \psi_{LT}) = \textbf{0.409}$

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Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance; $M_{y,Rk} = W_{pl.y} \times f_y = \textbf{87.9 kNm}$

Characteristic moment resistance; $M_{z,Rk} = W_{pl.z} \times f_y = \textbf{39.6 kNm}$

Characteristic resistance to normal force; $N_{Rk} = A \times f_y = \textbf{1358.3 kN}$

Interaction factors; $k_{yy} = C_{my} \times (1 + \min(\ \bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) = \textbf{0.412}$

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) = \textbf{0.983}$

Interaction formulae - eq 6.61 & eq 6.62; $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = \textbf{0.035}$

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) = \textbf{0.07}$

PASS - Combined bending and compression checks are satisfied

Consider Combination 5 - 1.0G + 1.0ψ_0Q + 0.5S + 1.0W (Service)

Check design 1017 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection; $\delta_y = \textbf{0.5 mm}$

Allowable deflection; $\delta_{y,Allowable} = L_{m3\_s1} / 180 = \textbf{16.7 mm}$

$\delta_y / \delta_{y,Allowable} = \textbf{0.028}$

PASS - Allowable deflection exceeds design deflection

;

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MK8 BEAM END CONNECTION DESIGN (BS5950)

BEAM TO COLUMN - END PLATE CONNECTION

TEDDS calculation version 2.0.16

Section Details

Column UC 152x152x30;; $Grade_{column}$ = **"S355"**

Beam UC 305x305x97;; $Grade_{beam}$ = **"S355"**

Endplate - 320 x 320 x 10;; $Grade_{endplate}$ = **"S355"**

Bolts M20 (Grade 8.8)

SECTION THROUGH SUPPORTING BEAM   SECTION A - A

Connection Details

;	number of bolt rows;	n_{bolts} = **2**
;	Bolt pitch;;	p_{bolts} = **100** mm
;	Bolt gauge;	g_{bolts} = **90** mm
;	End plate end distance (top & bottom);	$e_{1endplate}$ = **110** mm
	End plate edge distance;	$e_{2endplate}$ = ($d_{endplate}$ - g_{bolts}) / 2 = **115** mm
	End plate length;	$l_{endplate}$ = $p_{bolts} \times (n_{bolts}-1) + 2 \times e_{1endplate}$ = **320** mm
;	Weld leg length;	s_{weld} = **6** mm
;	Beam end reaction;	Q = **105.5** kN

Check 1 - Essential detailing requirements

;	End plate thickness;	$t_{endplate}$ = **10** mm; **PASS**
;	Bolt gauge;	g_{bolts} = **90** mm; **PASS**
;	End plate Length;	$l_{endplate}$ = **320** mm
;		**End plate length for torsional requirements : PASS**

Check 2 - Shear capacity of bolt group connecting end plate to supporting column

Shear capacity of top pair of bolts;;;;

$p_{bsendplate}$ = **550** N/mm²

$P_{sbolts1}$ = min(P_{sbolts}, 0.5 × $e_{1endplate}$ × $t_{endplate}$ × $p_{bsendplate}$) = **91.9** kN

Shear capacity of other bolts

P_{sbolts} = **91.9** kN

Shear capacity of bolt group - sum of bolt capacities

;$P_{sboltssum}$ = 2 × $P_{sbolts1}$ + 2 × (n_{bolts} - 1) × P_{sbolts} = **367.5** kN

;Shear on bolt group; Q = **105.5** kN

Utilisation factor; U_{check2} = Q / $P_{sboltssum}$ = **0.287**

Shear capacity of bolt group to column : PASS

Check 3 - Shear and bearing capacity of end plate

for shear

;;

$p_{yendplate}$ = **355** N/mm²

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;; $e_{1aendplate} = e_{1endplate} = $ **110** mm

;; $A_{vendplate} = 0.9 \times (2 \times e_{1aendplate} + (n_{bolts} - 1) \times p_{bolts}) \times t_{endplate} = $ **2880** mm^2

; $A_{vnetendplate} = A_{vendplate} - n_{bolts} \times D_{hbolts} \times t_{endplate} = $ **2440** mm^2

Effective net area coefficient

       $K_{eendplate} = $ **1.10**

Plain shear capacity of end plate

       $P_{vPendplate} = min(0.6 \times p_{yendplate} \times A_{vendplate}, 0.7 \times K_{eendplate} \times p_{yendplate} \times A_{vnetendplate}) = $ **613.4** kN

       $A_{v1endplate} = (e_{1aendplate} + (n_{bolts} - 1) \times p_{bolts}) \times t_{endplate} = $ **2100** mm^2

; $A_{teffendplate} = (e_{2endplate} - 0.5 \times D_{hbolts}) \times t_{endplate} = $ **1040** mm^2

Block shear capacity of end plate

       $P_{vBendplate} = 0.6 \times p_{yendplate} \times A_{v1endplate} + 0.6 \times K_{eendplate} \times p_{yendplate} \times A_{teffendplate} = $ **691.0** kN

Shear capacity of the end plate; $P_{vendplate} = min (P_{vPendplate}, P_{vBendplate}) = $ **613.4** kN

;Shear force on the end plate; $Q / 2 = $ **52.8** kN

Utilisation factor; $U_{check3shear} = Q / (2 \times P_{vendplate}) = $ **0.086**

Shear capacity of end plate : PASS

for bearing

       $e_{endplate} = e_{1endplate} = $ **110** mm

       bearing strength of the end plate

       $p_{bsendplate} = $ **550** N/mm^2

       For top bolt,
       bearing capacity of the end plate per bolt

       $P_{bsendplate1} = min(d_{bolts} \times t_{endplate} \times p_{bsendplate}, 0.5 \times e_{endplate} \times t_{endplate} \times p_{bsendplate}) = $ **110.0** kN

       For other bolts,
       bearing capacity of the end plate per bolt

       $P_{bsendplate} = d_{bolts} \times t_{endplate} \times p_{bsendplate} = $ **110.0** kN

Capacity of bolt group;

$P_{bsendplatesum} = 2 \times P_{bsendplate1} + 2 \times (n_{bolts} - 1) \times P_{bsendplate} = $ **440.0** kN

Bearing force on bolt group; $Q = $ **105.5** kN

Utilisation factor; $U_{check3bearing} = Q / P_{bsendplatesum} = $ **0.240**

Bearing capacity of endplate : PASS

Check 4 - shear capacity of the beam web at the endplate

;;

       $p_{ybeam} = $ **355** N/mm^2

;;;

;

       $A_{vbeam} = 0.9 \times min(l_{endplate}, d_{beam}) \times t_{beam} = $;**2198**; mm^2;

Shear capacity of beam web

       $P_{vbeam} = 0.6 \times p_{ybeam} \times A_{vbeam} = $ **468.2** kN

;Shear force on beam; $Q = $ **105.5** kN

Utilisation factor; $U_{check4shear} = Q / P_{vbeam} = $ **0.225**

Shear capacity of beam web : PASS

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Check 5 - capacity of the fillet welds connecting the end plate to the beam web

; Effective throat size of weld; $a_{weld} = s_{weld} \times 0.7 = $ **4.2** mm

;;;

Effective length of weld; $l_{weld} = 2 \times (min(l_{endplate}, 2 \times r_{beam} + d_{beam}) - 2 \times s_{weld}) = $;**530.2**; mm

; Design strength of weld; $p_{weld} = $ **220** N/mm^2

Capacity of fillet welds; $P_{weld} = p_{weld} \times l_{weld} \times a_{weld} = $ **489.9** kN

;Utilisation factor; $U_{check5weld} = Q / P_{weld} = $ **0.215**

Capacity of fillet weld : PASS

Check 6 - Local shear and bearing capacity of column flange

for shear

;; $e_{tbeama} = e_{tbeam} = $ **110** mm

;;; $A_{vcolumn} = (p_{bolts} + (n_{bolts} - 1) \times p_{bolts} + e_{tbeama}) \times T_{column} = $ **2914** mm^2

; $A_{vnetcolumn} = A_{vcolumn} - n_{bolts} \times D_{hbolts} \times T_{column} = $ **2500** mm^2

;

$p_{ycolumn} = $ **355** N/mm^2

Effective net area coefficient

$K_{ecolumn} = $ **1.10**

$P_{vcolumn} = min(0.6 \times p_{ycolumn} \times A_{vcolumn}, 0.7 \times K_{ecolumn} \times p_{ycolumn} \times A_{vnetcolumn}) = $ **620.7** kN

;Shear load on column; $Q / 2 = $ **52.8** kN

Utilisation factor; $U_{check6shear} = Q / (2 \times P_{vcolumn}) = $ **0.085**

Local shear capacity of column flange: PASS

for bearing

$p_{bscolumn} = $ **550** N/mm^2

$P_{bscolumn} = d_{bolts} \times T_{column} \times p_{bscolumn} = $ **103.4** kN

$Q / (2 \times n_{bolts}) = $ **26.4** kN

Utilisation factor; $U_{check6bearing} = Q / (2 \times n_{bolts} \times P_{bscolumn}) = $ **0.255**

Local bearing capacity of column flange : PASS

SUMMARY OF RESULTS

Check 2 - Shear capacity of bolt group connecting end plate to supporting column

Shear utilisation factor; $U_{check2} = $ **0.287**;   **PASS**

Check 3 - Capacity of end plate

Shear utilisation factor; $U_{check3shear} = $ **0.086**;   **PASS**

Bearing utilisation factor; $U_{check3bearing} = $ **0.240**;   **PASS**

Check 4 - Shear capacity of the beam web at the endplate

Shear utilisation factor; $U_{check4shear} = $ **0.225**;   **PASS**

Check 5 - Capacity of the fillet welds connecting the end plate to the beam web

Shear utilisation factor; $U_{check5weld} = $ **0.215**;   **PASS**

Check 6 - Local shear and bearing capacity of column flange

Shear utilisation factor; $U_{check6shear} = $ **0.085**;   **PASS**

Bearing utilisation factor; $U_{check6bearing} = $ **0.255**;   **PASS**

:

;

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BEAM END CONNECTION DESIGN (BS5950)

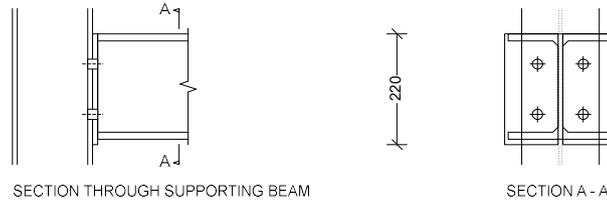### BEAM TO COLUMN - END PLATE CONNECTION

TEDDS calculation version 2.0.16

Section Details

Column UC 152x152x30;; $Grade_{column}$ = **"S355"**

Beam UC 203x203x60;; $Grade_{beam}$ = **"S355"**

Endplate - 220 x 220 x 10;; $Grade_{endplate}$ = **"S355"**

Bolts M20 (Grade 8.8)

SECTION THROUGH SUPPORTING BEAM   SECTION A - A

Connection Details

; number of bolt rows; n_{bolts} = **2**

; Bolt pitch;; p_{bolts} = **100** mm

; Bolt gauge; g_{bolts} = **90** mm

; End plate end distance (top & bottom); $e_{1endplate}$ = **60** mm

End plate edge distance; $e_{2endplate}$ = $(d_{endplate} - g_{bolts}) / 2$ = **65** mm

End plate length; $l_{endplate}$ = $p_{bolts} \times (n_{bolts}-1)+2 \times e_{1endplate}$ = **220** mm

; Weld leg length; s_{weld} = **6** mm

; Beam end reaction;   Q = **7.9** kN

Check 1 - Essential detailing requirements

; End plate thickness; $t_{endplate}$ = **10** mm; **PASS**

; Bolt gauge; g_{bolts} = **90** mm; **PASS**

; End plate Length; $l_{endplate}$ = **220** mm

;   **End plate length for torsional requirements : PASS**

Check 2 - Shear capacity of bolt group connecting end plate to supporting column

Shear capacity of top pair of bolts;;;;

$p_{bsendplate}$ = **550** N/mm^2

$P_{sbolts1}$ = $min(P_{sbolts}, 0.5 \times e_{1endplate} \times t_{endplate} \times p_{bsendplate})$ = **91.9** kN

Shear capacity of other bolts

P_{sbolts} = **91.9** kN

Shear capacity of bolt group - sum of bolt capacities

;$P_{sboltssum}$ = $2 \times P_{sbolts1} + 2 \times (n_{bolts} - 1) \times P_{sbolts}$ = **367.5** kN

;Shear on bolt group; Q = **7.9** kN

Utilisation factor; U_{check2} = $Q / P_{sboltssum}$ = **0.021**

Shear capacity of bolt group to column : PASS

Check 3 - Shear and bearing capacity of end plate

for shear

;;

$p_{yendplate}$ = **355** N/mm^2

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;; $e_{1aendplate} = e_{1endplate} = $ **60** mm

;; $A_{vendplate} = 0.9 \times (2 \times e_{1aendplate} + (n_{bolts} - 1) \times p_{bolts}) \times t_{endplate} = $ **1980** mm^2

; $A_{vnetendplate} = A_{vendplate} - n_{bolts} \times D_{hbolts} \times t_{endplate} = $ **1540** mm^2

Effective net area coefficient

$K_{eendplate} = $ **1.10**

Plain shear capacity of end plate

$P_{vPendplate} = min(0.6 \times p_{yendplate} \times A_{vendplate}, 0.7 \times K_{eendplate} \times p_{yendplate} \times A_{vnetendplate}) = $ **421.0** kN

$A_{v1endplate} = (e_{1aendplate} + (n_{bolts} - 1) \times p_{bolts}) \times t_{endplate} = $ **1600** mm^2

; $A_{teffendplate} = (e_{2endplate} - 0.5 \times D_{hbolts}) \times t_{endplate} = $ **540** mm^2

Block shear capacity of end plate

$P_{vBendplate} = 0.6 \times p_{yendplate} \times A_{v1endplate} + 0.6 \times K_{eendplate} \times p_{yendplate} \times A_{teffendplate} = $ **467.3** kN

Shear capacity of the end plate; $P_{vendplate} = min (P_{vPendplate}, P_{vBendplate}) = $ **421.0** kN

;Shear force on the end plate; Q / 2 = **4.0** kN

Utilisation factor; $U_{check3shear} = Q / (2 \times P_{vendplate}) = $ **0.009**

Shear capacity of end plate : PASS

for bearing

$e_{endplate} = e_{1endplate} = $ **60** mm

bearing strength of the end plate

$p_{bsendplate} = $ **550** N/mm^2

For top bolt,
bearing capacity of the end plate per bolt

$P_{bsendplate1} = min(d_{bolts} \times t_{endplate} \times p_{bsendplate}, 0.5 \times e_{endplate} \times t_{endplate} \times p_{bsendplate}) = $ **110.0** kN

For other bolts,
bearing capacity of the end plate per bolt

$P_{bsendplate} = d_{bolts} \times t_{endplate} \times p_{bsendplate} = $ **110.0** kN

Capacity of bolt group;

$P_{bsendplatesum} = 2 \times P_{bsendplate1} + 2 \times (n_{bolts} - 1) \times P_{bsendplate} = $ **440.0** kN

Bearing force on bolt group; Q = **7.9** kN

Utilisation factor; $U_{check3bearing} = Q / P_{bsendplatesum} = $ **0.018**

Bearing capacity of endplate : PASS

Check 4 - shear capacity of the beam web at the endplate

;;

$p_{ybeam} = $ **355** N/mm^2

;;;

;

$A_{vbeam} = 0.9 \times min(l_{endplate}, d_{beam}) \times t_{beam} = $;**1360**; mm^2;

Shear capacity of beam web

$P_{vbeam} = 0.6 \times p_{ybeam} \times A_{vbeam} = $ **289.8** kN

;Shear force on beam; Q = **7.9** kN

Utilisation factor; $U_{check4shear} = Q / P_{vbeam} = $ **0.027**

Shear capacity of beam web : PASS

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Check 5 - capacity of the fillet welds connecting the end plate to the beam web

; Effective throat size of weld; $a_{weld} = s_{weld} \times 0.7 = $ **4.2** mm

;;;

Effective length of weld; $l_{weld} = 2 \times (min(l_{endplate}, 2 \times r_{beam} + d_{beam}) - 2 \times s_{weld}) = $;**338.4**; mm

; Design strength of weld; $p_{weld} = $ **220** N/mm^2

Capacity of fillet welds; $P_{weld} = p_{weld} \times l_{weld} \times a_{weld} = $ **312.7** kN

;Utilisation factor; $U_{check5weld} = Q / P_{weld} = $ **0.025**

Capacity of fillet weld : PASS

Check 6 - Local shear and bearing capacity of column flange

for shear

;; $e_{tbeama} = e_{tbeam} = $ **60** mm

;;; $A_{vcolumn} = (p_{bolts} + (n_{bolts} - 1) \times p_{bolts} + e_{tbeama}) \times T_{column} = $ **2444** mm^2

; $A_{vnetcolumn} = A_{vcolumn} - n_{bolts} \times D_{hbolts} \times T_{column} = $ **2030** mm^2

;

$p_{ycolumn} = $ **355** N/mm^2

Effective net area coefficient

$K_{ecolumn} = $ **1.10**

$P_{vcolumn} = min(0.6 \times p_{ycolumn} \times A_{vcolumn}, 0.7 \times K_{ecolumn} \times p_{ycolumn} \times A_{vnetcolumn}) = $ **520.6** kN

;Shear load on column; $Q / 2 = $ **4.0** kN

Utilisation factor; $U_{check6shear} = Q / (2 \times P_{vcolumn}) = $ **0.008**

Local shear capacity of column flange: PASS

for bearing

$p_{bscolumn} = $ **550** N/mm^2

$P_{bscolumn} = d_{bolts} \times T_{column} \times p_{bscolumn} = $ **103.4** kN

$Q / (2 \times n_{bolts}) = $ **2.0** kN

Utilisation factor; $U_{check6bearing} = Q / (2 \times n_{bolts} \times P_{bscolumn}) = $ **0.019**

Local bearing capacity of column flange : PASS

SUMMARY OF RESULTS

Check 2 - Shear capacity of bolt group connecting end plate to supporting column

Shear utilisation factor; $U_{check2} = $ **0.021**; **PASS**

Check 3 - Capacity of end plate

Shear utilisation factor; $U_{check3shear} = $ **0.009**; **PASS**

Bearing utilisation factor; $U_{check3bearing} = $ **0.018**; **PASS**

Check 4 - Shear capacity of the beam web at the endplate

Shear utilisation factor; $U_{check4shear} = $ **0.027**; **PASS**

Check 5 - Capacity of the fillet welds connecting the end plate to the beam web

Shear utilisation factor; $U_{check5weld} = $ **0.025**; **PASS**

Check 6 - Local shear and bearing capacity of column flange

Shear utilisation factor; $U_{check6shear} = $ **0.008**; **PASS**

Bearing utilisation factor; $U_{check6bearing} = $ **0.019**; **PASS**

;

;