Appendix A

Structural reports and cantilever exclusion plan

RFT supporting information | Tender information P01 | 21 October 2026

This appendix brings together the Buildings B and D cantilever exclusion plan and the complete structureHaus structural reports for Buildings A, B, C and D. Tenderers should read the relevant building report, including its calculations, assumptions, exclusions and supporting information.

Contents

Section	Document	Starts	Pages
A1	B and D cantilever exclusion plan	2	1
A2	Building A structural report	3	45
A3	Building B structural report	48	45
A4	Building C structural report	93	44
A5	Building D structural report	137	38

How to use this appendix

Cantilever exclusion

No PV is to be installed in the red-outlined cantilever areas shown in A1 for Buildings B and D. The building-specific reports also identify these exclusions.

Assessment scope

The reports assess the existing building structure for the proposed PV loading. Their conclusions must be read with the stated limitations; they do not establish the adequacy of the PV-to-roof fixing system.

Document control

All four report letters are dated 21 January 2026. Report pages are reproduced in full and unchanged, including original pagination and blank pages. Contents page numbers refer to this combined PDF.

Report identification

The supplied Building A and Building D reports both carry reference 07229E_sH_SE_RP_004. Identify each report by its building title and its position in this appendix.

Cantilever areas excluded from PV installation

Red outlines identify the excluded areas. Read with the Building B and Building D structural reports.

BUILDING B



BUILDING D



Source graphic: PV Plan for Buildings B & D - no PV on the canterleaver.pdf

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Our ref: 07229E_sH_SE_RP_004

Dr Sally Murrall-Smith PhD
Totnes Renewable Energy Society
107 High Street
Totnes
TQ9 5SN

21st January 2026

Dear Sally,

Structural Check at Dart Marine Park, Steamer Quay Road, Totnes, TQ9 5AL – Building A

As requested, and commissioned, we have carried out a structural survey of the portal frame building at Dart Marine Park, carried out calculations and prepared this report, for the purpose of justifying the proposed PV panel installation to the roof.

The building is of a two storey portalised steel frame, we could not measure the floor steel due to the finishes of the building, we requested 'As-Built' drawings, but these haven't been available. However, we are confident that the compressive utilisation of the steel columns is not at question, more the overall stability, as such we have included a tie beam at floor level but not analysed this element.

The portal steel frame has 203 x 133 x 30 UB Rafters and 203 UC 46 Columns, the portal frames are at 5.7 metre centres. The dimension of the portal frame is shown in the calculations within this report.

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The proposed PV system is fixed to the roof, loadings are taken as 0.15kN/m^2 which is applied on top of the snow loading at 0.6kN/m^2 and the dead weight loading of 0.5kN/m^2 including services.

Structural calculations have been carried out to check the capacity of these members, and determine adequacy of the existing structure for the proposed PV panel instillation.

We are pleased to confirm that the steel frame member sizes are adequate to support the additional vertical loading of the proposed installation.

Therefore, we can confirm that installing the new roof panels upon the existing roof of this structure will not have a structurally detrimental effect.

This letter confirms cover under our PI should any failure of the steel frame occur due to inadequate capacity.

This letter is not intended to and does not relate to the adequacy or assessment of the fixing system to the Photovoltaics to the roof.

Yours sincerely,



Mark Harris MIStructE
Director
structureHaus Ltd

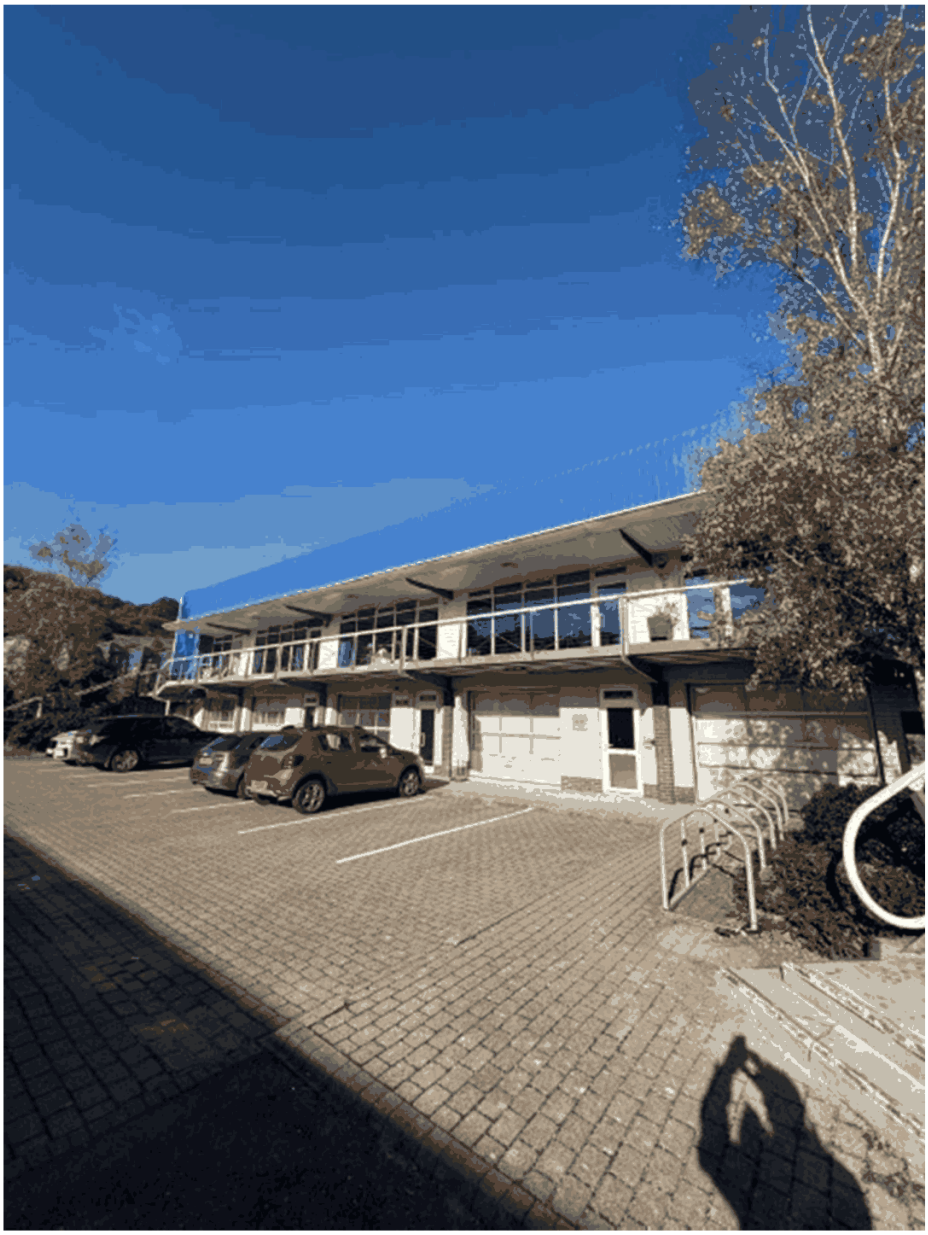
Appendix A – Information Provided and Photos

Appendix B – Structural Calculations

APPENDIX A – INFORMATION PROVIDED & SITE PHOTOS



Proposed PV layout



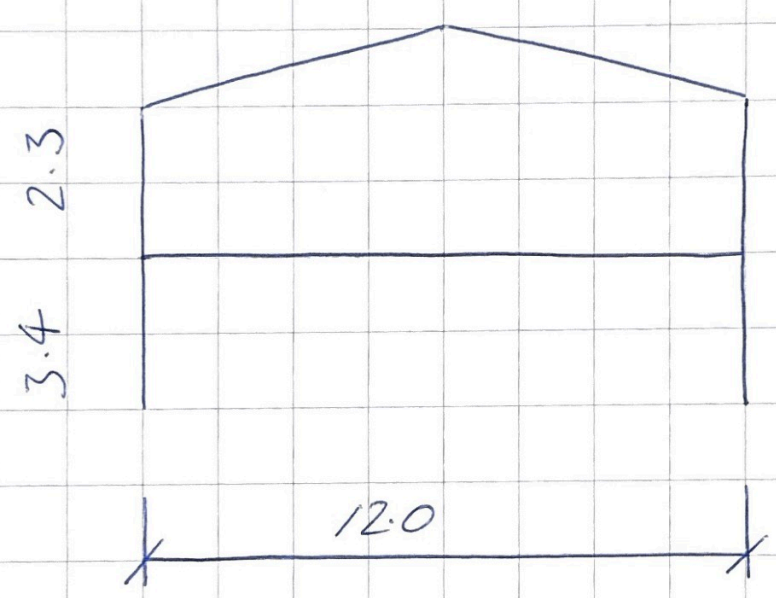
Site Visit Photo



Site Visit Photo

APPENDIX B – STRUCTURAL CALCULATIONS

Building A



@ 5.7 m c/c's

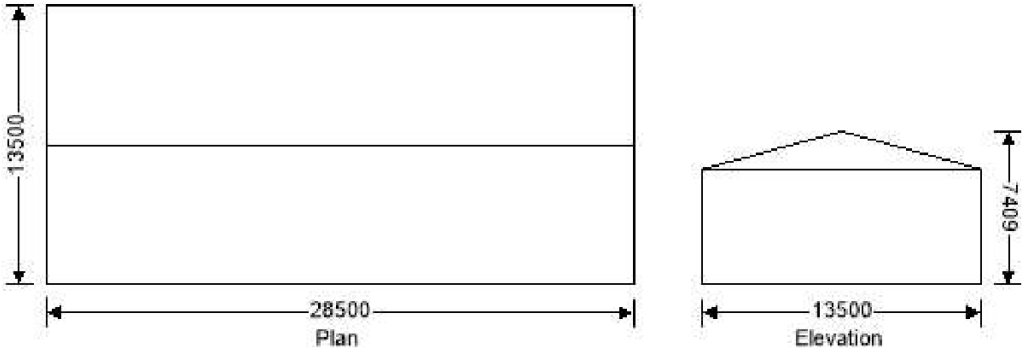
Rafters 203 × 133 × 30 UB

Columns 203 UC 46

WIND LOADING (BS6399)

In accordance with BS6399

Tedds calculation version 3.0.19



Building data

Type of roof	Duopitch
Length of building	L = **28500** mm
Width of building	W = **13500** mm
Height to eaves	H = **5600** mm
Pitch of roof	α_0 = **15.0** deg
Reference height	H_r = **7409** mm

Dynamic classification

Building type factor (Table 1)	K_b = **1.0**
Dynamic augmentation factor (1.6.1)	$C_r = [K_b \times (H_r / (0.1\ m))^{0.75}] / (800 \times \log(H_r / (0.1\ m))) =$ **0.02**

Site wind speed

Location	Totnes
Basic wind speed (Figure 6 BS6399:Pt 2)	V_b = **22.5** m/s
Site altitude	Δs = **1** m
Upwind distance from sea to site	d_{sea} = **10** km
Direction factor	S_d =**0.82**
Seasonal factor	S_s = **1.00**
Probability factor	S_p = **1.00**
Critical gap between buildings	g = **5000** mm
Topography not significant	
Altitude factor	$S_a = 1 + 0.001 \times \Delta s$ / 1m = **1.00**
Site wind speed	$V_s = V_b \times S_a \times S_d \times S_s \times S_p$ = **18.5** m/s
Terrain category	Sea
Displacement height (sheltering effect excluded)	H_d = 0mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

Dynamic pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought)	H_{ref} = **5600**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **5600**mm
Fetch factor (Table 22)	S_c = **0.959**

Turbulence factor (Table 22)	S_t = **0.190**
Gust peak factor	g_t = **3.44**
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.59**
Effective wind speed	$V_e = V_s \times S_b$ = **29.3** m/s
Dynamic pressure	q_s = 0.613 kg/m^3 × V_e^2 = **0.527** kN/m^2

Dynamic pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought)	H_{ref} = **7409**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **7409**mm
Fetch factor (Table 22)	S_c = **1.005**
Turbulence factor (Table 22)	S_t = **0.185**
Gust peak factor	g_t = **3.44**
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.64**
Effective wind speed	$V_e = V_s \times S_b$ = **30.4** m/s
Dynamic pressure	q_s = 0.613 kg/m^3 × V_e^2 = **0.566** kN/m^2

Size effect factors

Diagonal dimension for gablewall	a_{eg} = **14.6** m
External size effect factor gablewall	C_{aeg} = **0.903**
Diagonal dimension for side wall	a_{es} = **29.0** m
External size effect factor side wall	C_{aes} = **0.841**
Diagonal dimension for roof	a_{er} = **29.3** m
External size effect factor roof	C_{aer} = **0.840**
Room/storey volume for internal size effect factor	V_i = **1740.000** m^3
Diagonal dimension for internal size effect factors	$a_i = 10 \times (V_i)^{1/3}$ = **120.277** m
Internal size effect factor	C_{ai} = **0.712**

Pressures and forces

Net pressure	$p = q_s \times c_{pe} \times C_{ae} - q_s \times c_{pi} \times C_{ai}$
Net force	$F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.10	0.57	0.840	-0.60	22.73	-13.71
B (-ve)	-0.80	0.57	0.840	-0.46	20.99	-9.67
C (-ve)	-0.40	0.57	0.840	-0.27	155.44	-42.06
E (-ve)	-1.30	0.57	0.840	-0.70	22.73	-15.87
F (-ve)	-0.90	0.57	0.840	-0.51	20.99	-10.66
G (-ve)	-0.50	0.57	0.840	-0.32	155.44	-49.44

Total vertical net force	$F_{w,v}$ = **-136.59** kN
Total horizontal net force	$F_{w,h}$ = **2.73** kN

Walls load case 1 - Wind 0, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.40	0.57	0.903	-0.80	17.77	-14.19
B	-0.83	0.57	0.903	-0.51	70.04	-35.51
w	0.73	0.53	0.841	0.25	159.60	39.78
l	-0.50	0.53	0.841	-0.30	159.60	-47.29

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-47.3** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **39.8** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **77.6** kN

Roof load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A (+ve)	0.20	0.57	0.840	0.22	22.73	4.91
B (+ve)	0.20	0.57	0.840	0.22	20.99	4.53
C (+ve)	0.20	0.57	0.840	0.22	155.44	33.55
E (+ve)	-1.30	0.57	0.840	-0.50	22.73	-11.29
F (+ve)	-0.90	0.57	0.840	-0.31	20.99	-6.44
G (+ve)	-0.50	0.57	0.840	-0.12	155.44	-18.14

Total vertical net force $F_{w,v}$ = **6.87** kN

Total horizontal net force $F_{w,h}$ = **20.41** kN

Walls load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.40	0.57	0.903	-0.60	17.77	-10.61
B	-0.83	0.57	0.903	-0.31	70.04	-21.41
w	0.73	0.53	0.841	0.44	159.60	69.70
l	-0.50	0.53	0.841	-0.11	159.60	-17.38

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-17.4** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **69.7** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **92.9** kN

Roof load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.60	0.57	0.840	-0.84	9.43	-7.93
B (-ve)	-1.50	0.57	0.840	-0.79	9.43	-7.48
C (-ve)	-0.60	0.57	0.840	-0.37	75.47	-27.59
D (-ve)	-0.40	0.57	0.840	-0.27	303.98	-82.25

Total vertical net force $F_{w,v}$ = **-120.99** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.54	0.53	0.841	-0.75	12.54	-9.47
B	-0.88	0.53	0.841	-0.46	50.18	-23.28
C	-0.81	0.53	0.841	-0.44	96.88	-42.19
w	0.61	0.57	0.903	0.23	87.81	20.41
l	-0.50	0.57	0.903	-0.34	87.81	-29.50

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-29.5** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **20.4** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **43.1** kN

Roof load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.60	0.57	0.840	-0.64	9.43	-6.03
B (-ve)	-1.50	0.57	0.840	-0.59	9.43	-5.58
C (-ve)	-0.60	0.57	0.840	-0.16	75.47	-12.39
D (-ve)	-0.40	0.57	0.840	-0.07	303.98	-21.04

Total vertical net force $F_{w,v}$ = **-43.51** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

structureHaus Ltd	Project: Dart Marine Park, Totnes				Job no.: 07229E	
	Calcs for: Wind				Start page no./Revision: 6	
	Calcs by: MDH	Calcs date: 11/11/2025	Checked by: DALP	Checked date:	Approved by:	Approved date:

Walls load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

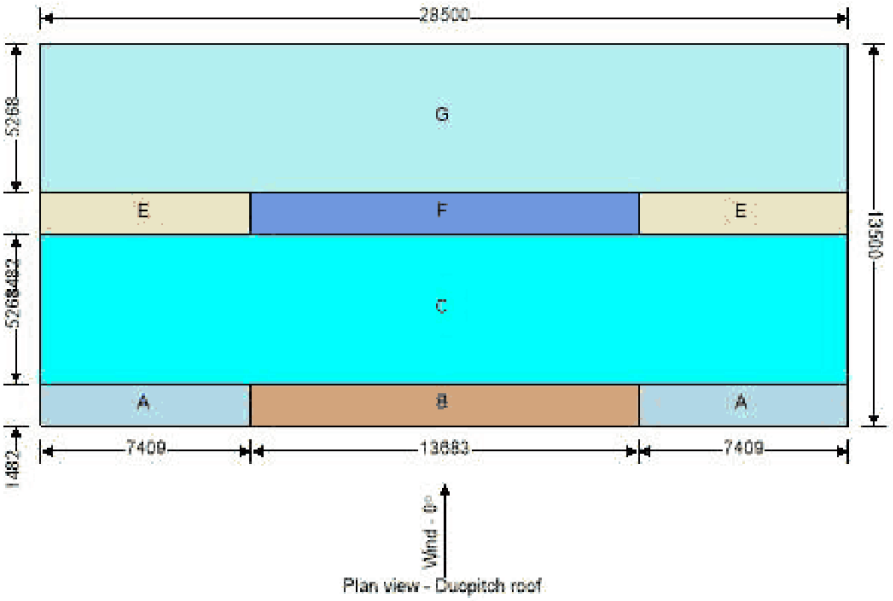
Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.54	0.53	0.841	-0.57	12.54	-7.12
B	-0.88	0.53	0.841	-0.28	50.18	-13.87
C	-0.81	0.53	0.841	-0.25	96.88	-24.03
w	0.61	0.57	0.903	0.43	87.81	38.09
l	-0.50	0.57	0.903	-0.13	87.81	-11.82

Overall loading

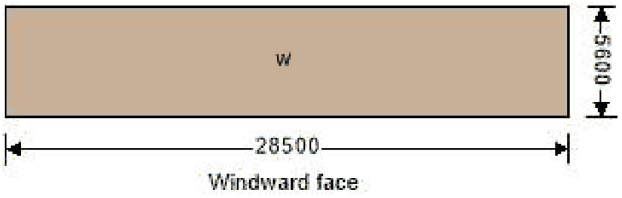
Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-11.8** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **38.1** kN

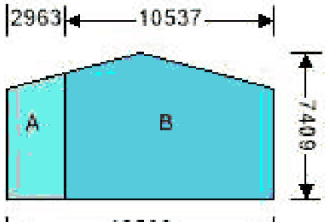
Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **43.1** kN



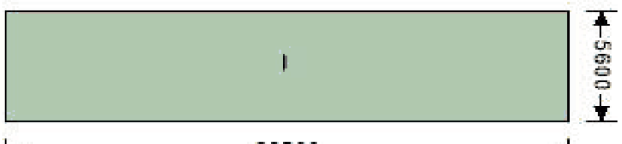
Plan view - Duopitch roof



Windward face

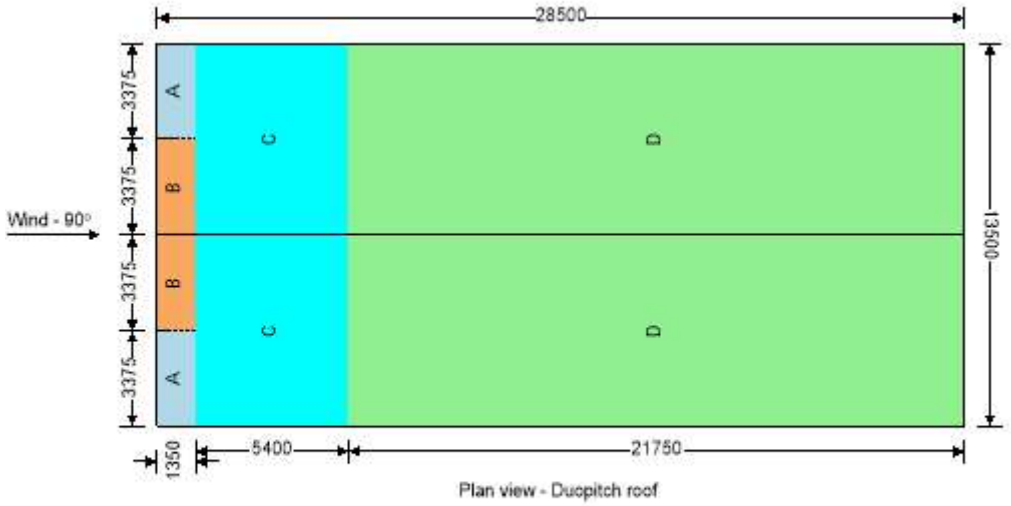


Side face

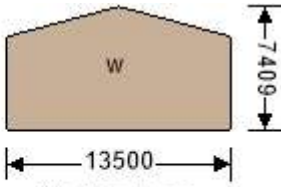


Leeward face

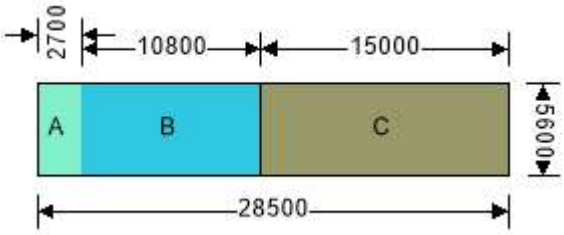
structureHaus Ltd	Project: Dart Marine Park, Totnes				Job no.: 07229E	
	Calcs for: Wind				Start page no./Revision: 7	
	Calcs by: MDH	Calcs date: 11/11/2025	Checked by: DALP	Checked date:	Approved by:	Approved date:



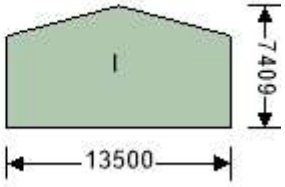
Plan view - Duopitch roof



Windward face

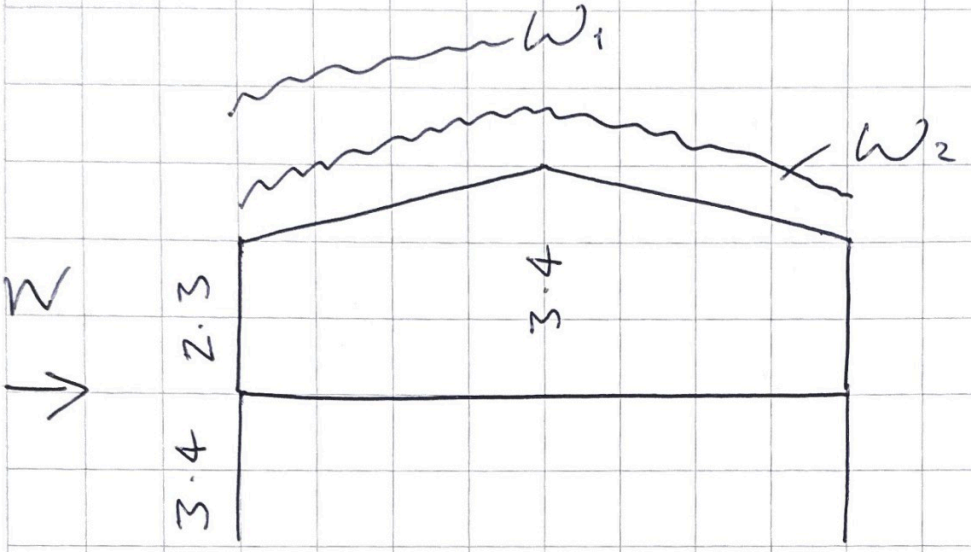


Side face



Leeward face

Building A



@ 5.7m c/c's

$W = 1.12 \times 5.7 = 6.38\ kN/m$

$W_1 = 0.12 \times 5.7 = 0.68\ kN/m$

$W_2 = 0.5 \times 5.7 = 2.85\ kN/m$

$0.6 = 3.42\ kN/m$

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

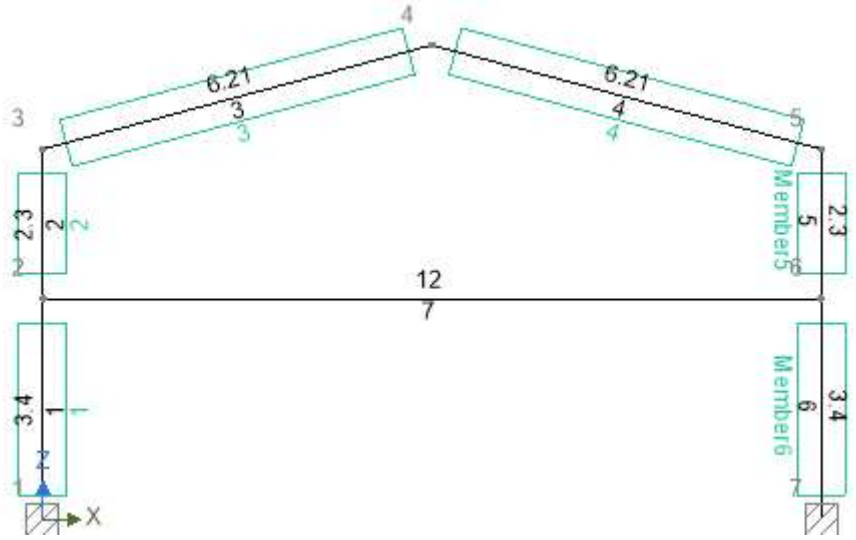
Tedds calculation version 4.5.06

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (m) - Steel (EC3)



Materials

Name	Density (kg/m³)	Youngs Modulus kN/mm²	Shear Modulus kN/mm²	Thermal Coefficient °C⁻¹
Steel (EC3)	7850	210	80.8	0.000012

Sections

Name	Area (cm²)	Moment of inertia Major (cm⁴)	Moment of inertia Minor (cm⁴)	Shear area parallel to Minor (cm²)	Shear area parallel to Major (cm²)
UKC 203x203x46	58.7	4567.8	1548.2	14.6	40.3
UB 203x133x30	38.2	2895.6	384.7	13.2	23.1

Nodes

Node	Co-ordinates X (m)	Co-ordinates Z (m)	Freedom X	Freedom Z	Freedom Rot.	Coordinate system Name	Coordinate system Angle (°)	Spring X (kN/m)	Spring Z (kN/m)	Spring Rot. kNm/°
1	0	0	Fixed	Fixed	Fixed		0	0	0	0
2	0	3.4	Free	Free	Free		0	0	0	0
3	0	5.7	Free	Free	Free		0	0	0	0
4	6	7.3	Free	Free	Free		0	0	0	0
5	12	5.7	Free	Free	Free		0	0	0	0
6	12	3.4	Free	Free	Free		0	0	0	0
7	12	0	Fixed	Fixed	Fixed		0	0	0	0

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	3.4	1	2	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
2	2.3	2	3	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
3	6.21	3	4	UB 203x133x30	Steel (EC3)	Fixed	Fixed	Fixed	
4	6.21	4	5	UB 203x133x30	Steel (EC3)	Fixed	Fixed	Fixed	
5	2.3	5	6	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
6	3.4	6	7	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
7	12	2	6	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	

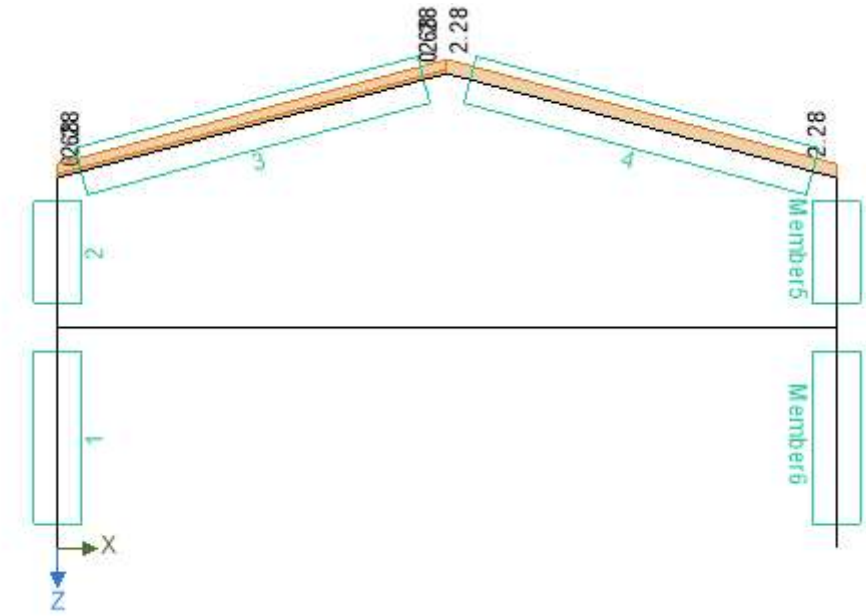
Members

Name	Elements	
	Start	End
1	1	1
2	2	2
3	3	3
4	4	4
Member5	5	5
Member6	6	6

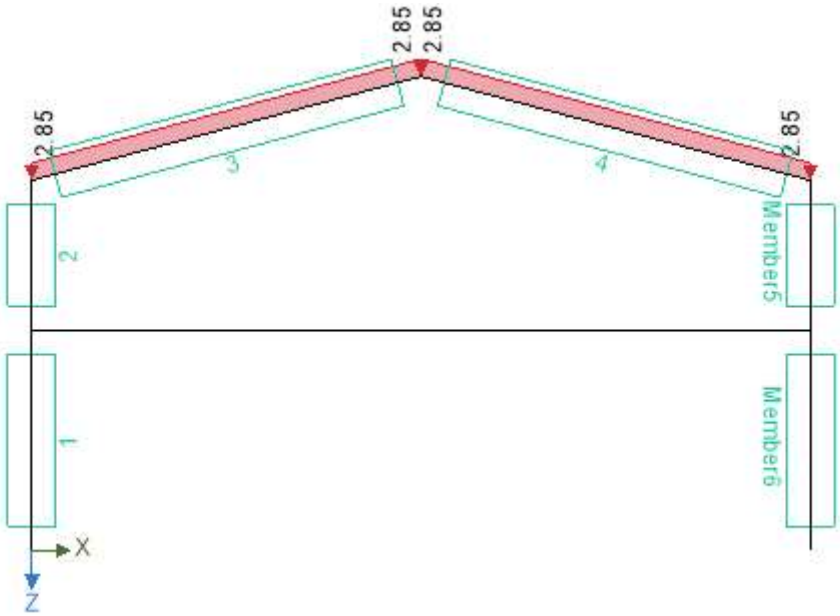
Loading

Self weight included

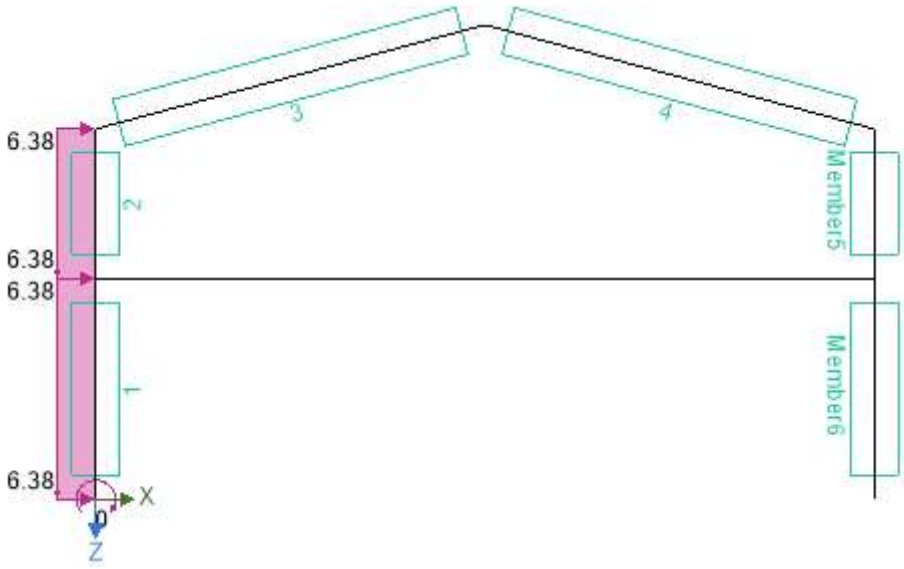
Permanent - Loading (kN/m)



Imposed - Loading (kN/m)



Wind - Loading (kN/m,kNm)



Load combination factors

Load combination	Self Weight	Permanent	Imposed	Wind
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50	
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00	
1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	1.50	0.75
1.0G + 1.0Q + 0.5S + 0.5W (Service)	1.00	1.00	1.00	0.50

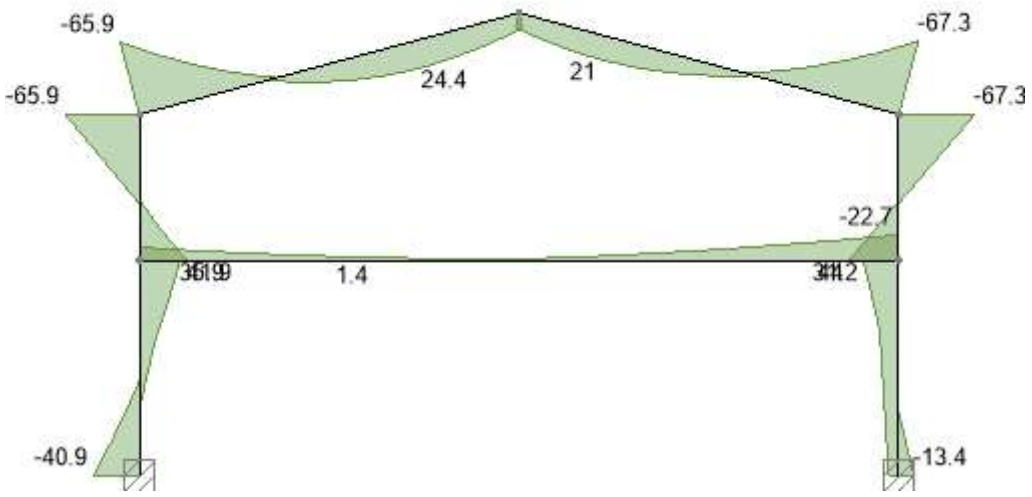
Member Loads

Member	Load case	Load Type	Orientation	Description
3	Permanent	UDL	GlobalZ	0.68 kN/m
3	Permanent	UDL	GlobalZ	2.28 kN/m
4	Permanent	UDL	GlobalZ	2.28 kN/m
3	Imposed	UDL	GlobalZ	2.85 kN/m
4	Imposed	UDL	GlobalZ	2.85 kN/m
1	Wind	Point Moment	GlobalY	0 kNm at 0 m
1	Wind	UDL	GlobalX	6.38 kN/m
2	Wind	UDL	GlobalX	6.38 kN/m

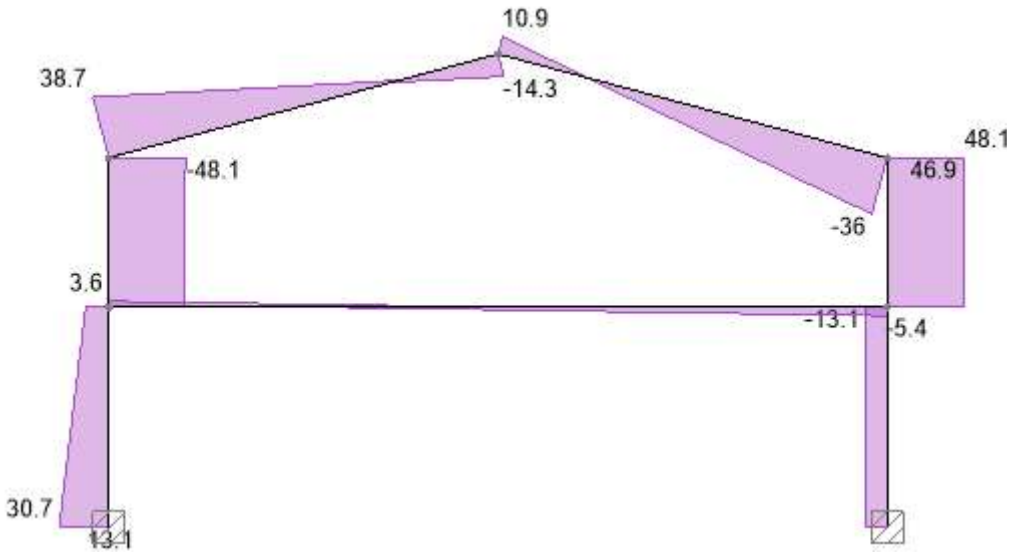
Results

Forces

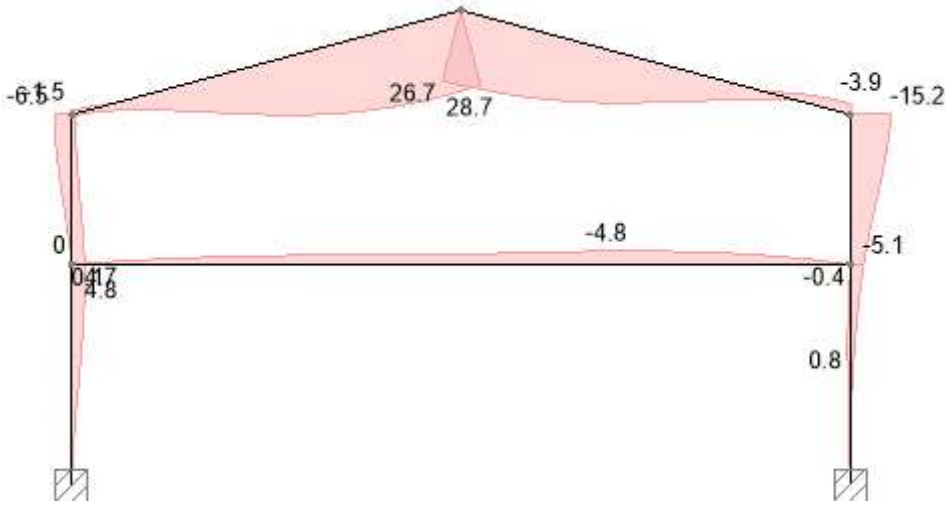
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



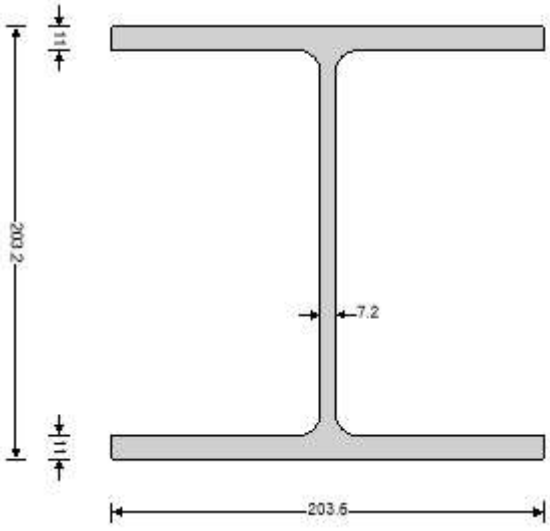
Partial factors - Section 6.1

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**

1 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	t_{nom} = max(t_f, t_w) = **11** mm
Nominal yield strength	f_y = **355** N/mm²
Nominal ultimate tensile strength	f_u = **470** N/mm²
Modulus of elasticity	E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, i_y, 88.19 mm
Radius of gyration about z-axis, i_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230864 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section

$c = d =$ **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.570**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$c = (b - t_w - 2 \times r) / 2 =$ **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force — $N_{Ed} =$ **57.2** kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **2085** kN

$N_{Ed} / N_{c,Rd} =$ **0.027**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length — $L_{cr,y} = L_{m1\_s1} =$ **3400** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **8189.7** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.505**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — b

Imperfection factor - Table 6.1 — $\alpha_y =$ **0.34**

Buckling reduction determination factor — $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.679**

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.882**

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **1839.3** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.031**

PASS - Design buckling resistance exceeds design compression

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

Critical buckling length — $L_{cr,z} = L_{m1\_s1\_seg1} =$ **3400** mm

Critical buckling force — $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **2775.8** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **0.867**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — c

Imperfection factor - Table 6.1 — $\alpha_z =$ **0.49**

Buckling reduction determination factor — $\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **1.039**

Buckling reduction factor - eq 6.49 — $\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1) =$ **0.62**

Design buckling resistance - eq 6.47 — $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **1293.6** kN

$N_{Ed} / N_{b,z,Rd} =$ **0.044**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length $L_{cr,T} = L_{m1\_s1\_seg1\_R}$ = **3400** mm

Distance from shear centre to centroid in y axis y_0 = **0.0** mm

Distance from shear centre to centroid in z axis z_0 = **0.0** mm

Radius of gyration $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **102.0** mm

Elastic critical torsional buckling force $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **4180** kN

Torsion factor $\beta_T = 1 - (y_0 / i_0)^2$ = **1**

Elastic critical torsional-flexural buckling force $N_{cr,TF}$ = **4180** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.706**

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

Buckling curve - Table 6.2 c

Imperfection factor - Table 6.1 α_T = **0.49**

Buckling reduction determination factor $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.873**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.721**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **1502.9** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.038**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **181.2** mm η = **1.000**

$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **30.7** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.088**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **40.9** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.231**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 k_c = **0.634**

$C_1 = 1 / k_c^2$ = **2.49**

Poissons ratio ν = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m1\_s1\_seg1\_B}$ = **3400** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **865.6** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.452**

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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.585**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.980**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.861**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.231**

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
	$N_{Ed} / N_{y,lim}$ = **0.247**

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	ψ_y = 35.916 kNm / -40.856 kNm = **-0.879**
	α_y = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 35.916 kNm / -40.856 kNm = **-0.879**
	α_{LT} = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.404**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.974**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.126**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.274**

PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm η = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{y,Ed}$ = **14.4** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm^2
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.042**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **35.9** kNm

Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.203**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.634**
	$C_1 = 1 / k_c^2$ = **2.49**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2
Unrestrained effective length	$L = 1.0 \times L_{m1\_s1\_seg1\_T}$ = **3400** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **865.6** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.452**
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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.585**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.980**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.861**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.203**
	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	ψ_y = 35.916 kNm / -40.856 kNm = **-0.879**
	α_y = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 35.916 kNm / -40.856 kNm = **-0.879**
	α_{LT} = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.404**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.975**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.114**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.245**
	PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design 3199 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection — δ_y = **4.8** mm

Allowable deflection — $\delta_{y,Allowable} = L_{m1\_s1} / 100$ = **34** mm

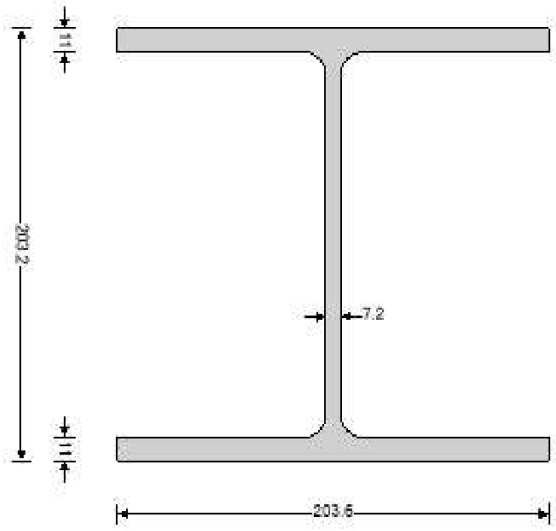
$\delta_y / \delta_{y,Allowable}$ = **0.14**

PASS - Allowable deflection exceeds design deflection

2 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **11** mm
Nominal yield strength	f_y = **355** N/mm²
Nominal ultimate tensile strength	f_u = **470** N/mm²
Modulus of elasticity	E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, i_y, 88.19 mm
Radius of gyration about z-axis, i_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230884 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section — c = d = **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.566**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force	N_{Ed} = **54** kN
Design resistance of section - eq 6.10	$N_{c,Rd} = A \times f_y / \gamma_{M0}$ = **2085** kN
	$N_{Ed} / N_{c,Rd}$ = **0.026**
	PASS - Design compression resistance exceeds design compression

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

Critical buckling length	$L_{cr,y} = L_{m2\_s1}$ = **2300** mm
Critical buckling force	$N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **17896.7** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.341**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	α_y = **0.34**
Buckling reduction determination factor	$\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.582**
Buckling reduction factor - eq 6.49	$\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.949**
Design buckling resistance - eq 6.47	$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **1978.1** kN
	$N_{Ed} / N_{b,y,Rd}$ = **0.027**
	PASS - Design buckling resistance exceeds design compression

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

Critical buckling length	$L_{cr,z} = L_{m2\_s1\_seg1}$ = **2300** mm
Critical buckling force	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **6065.9** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **0.586**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_z = **0.49**
Buckling reduction determination factor	$\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **0.766**
Buckling reduction factor - eq 6.49	$\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.793**
Design buckling resistance - eq 6.47	$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **1654.4** kN
	$N_{Ed} / N_{b,z,Rd}$ = **0.033**
	PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m2\_s1\_seg1\_R}$ = **2300** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **102.0** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **7097.8** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **4180** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.706**

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

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_T = **0.49**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.873**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.721**

Design buckling resistance - eq 6.47

$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **1502.9** kN

$N_{Ed} / N_{b,T,Rd} =$ **0.036**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web

$h_w = h - 2 \times t_f =$ **181.2** mm $\eta =$ **1.000**

$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force

$V_{y,Ed} =$ **46.9** kN

Shear area - cl 6.2.6(3)

$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1698** mm²

Design shear resistance - cl 6.2.6(2)

$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **347.9** kN

$V_{y,Ed} / V_{c,y,Rd} =$ **0.135**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$M_{y,Ed} =$ **41.9** kNm

Design bending resistance moment - eq 6.13

$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **176.6** kNm

$M_{y,Ed} / M_{c,y,Rd} =$ **0.237**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$k_c =$ **0.65**

$C_1 = 1 / k_c^2 =$ **2.367**

Poissons ratio

$\nu =$ **0.3**

Shear modulus

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

Unrestrained effective length

$L = 1.0 \times L_{m2\_s1\_seg1\_T} =$ **2300** mm

Elastic critical buckling moment

$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **1584.8** kNm

Slenderness ratio for lateral torsional buckling

$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} =$ **0.334**

Limiting slenderness ratio

$\bar{\lambda}_{LT,0} =$ **0.4**

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

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

$N_{Ed} / N_{y,lim} =$ **0.233**

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

$\psi_y = 41.933$ kNm / -65.866 kNm = **-0.637**

$\alpha_y = -11.966$ kNm / -65.866 kNm = **0.182**

$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4) =$ **0.400**

$\psi_{LT} = 41.933$ kNm / -65.866 kNm = **-0.637**

$\alpha_{LT} = -11.966$ kNm / -65.866 kNm = **0.182**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance

$M_{y,Rk} = W_{pl,y} \times f_y =$ **176.6** kNm

Characteristic moment resistance

$M_{z,Rk} = W_{pl,z} \times f_y =$ **82** kNm

Characteristic resistance to normal force

$N_{Rk} = A \times f_y =$ **2085** kN

Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.402**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.987**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.123**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.267**
	PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f =$ **181.2** mm $\quad \eta =$ **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed} =$ **46.9** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1698** mm^2
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd} =$ **0.135**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed} =$ **65.9** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd} =$ **0.373**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	$k_c =$ **0.65**
	$C_1 = 1 / k_c^2 =$ **2.367**
Poissons ratio	$\nu =$ **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)] =$ **80769** N/mm^2
Unrestrained effective length	$L = 1.0 \times L_{m2\_s1\_seg1\_B} =$ **2300** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **1584.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} =$ **0.334**
Limiting slenderness ratio	$\bar{\lambda}_{LT,0} =$ **0.4**
	$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3	$\psi_y =$ 41.933 kNm / -65.866 kNm = **-0.637**
	$\alpha_y =$ -11.966 kNm / -65.866 kNm = **0.182**
	$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4) =$ **0.400**
	$\psi_{LT} =$ 41.933 kNm / -65.866 kNm = **-0.637**
	$\alpha_{LT} =$ -11.966 kNm / -65.866 kNm = **0.182**
	$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y =$ **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y =$ **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y =$ **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.402**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.988**

Interaction formulae - eq 6.61 & eq 6.62

$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.176**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.4**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection — $\delta_y =$ **4.7** mm

Allowable deflection — $\delta_{y,Allowable} = L_{m2\_s1} / 100 =$ **23** mm

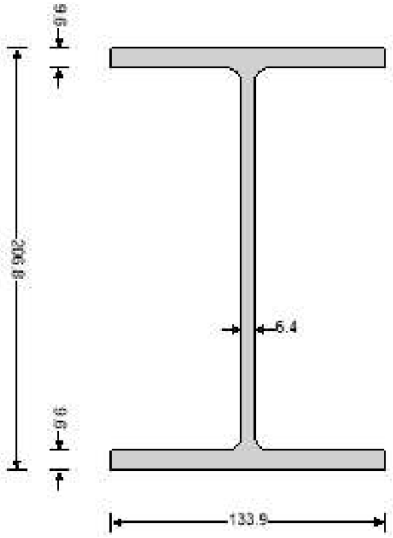
$\delta_y / \delta_{y,Allowable} =$ **0.205**

PASS - Allowable deflection exceeds design deflection

3 design

Section details

Section type	UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_f, t_w) =$ **9.6** mm
Nominal yield strength	$f_y =$ **275** N/mm²
Nominal ultimate tensile strength	$f_u =$ **410** N/mm²
Modulus of elasticity	E = **210000** N/mm²



UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 206.8 mm
Section breadth, b, 133.9 mm
Flange thickness, t_f, 9.6 mm
Web thickness, t_w, 6.4 mm
Root radius, r, 7.6 mm
Area of section, A, 3821 mm²
Radius of gyration about y-axis, i_y, 87.051 mm
Radius of gyration about z-axis, i_z, 31.728 mm
Elastic section modulus about y-axis, $W_{el,y}$, 280036 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 57454 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 314366 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 88224 mm³
Second moment of area about y-axis, I_y, 28955684 mm⁴
Second moment of area about z-axis, I_z, 3846542 mm⁴

Lateral restraint

Upper flange has full lateral restraint

Lower flange has lateral restraint at supports plus 1200 & 6000 mm

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} =$ **0.92**

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

Width of section — c = d = **172.4** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.597**

$c / t_w = 26.9 = 29.1 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **56.1** mm

$c / t_f = 5.8 = 6.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force $N_{Ed} =$ **58.8** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **1050.8** kN

$N_{Ed} / N_{c,Rd} =$ **0.056**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length $L_{cr,y} = L_{m3\_s1} =$ **6210** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **1556.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.822**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_y =$ **0.21**

Buckling reduction determination factor $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.903**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.783**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **822.9** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.071**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length $L_{cr,T} = L_{m3\_s1\_seg1\_R} =$ **1200** mm

Distance from shear centre to centroid in y axis $y_0 =$ **0.0** mm

Distance from shear centre to centroid in z axis $z_0 =$ **0.0** mm

Radius of gyration $i_0 = \sqrt{(i_y^2 + i_z^2)} =$ **92.7** mm

Elastic critical torsional buckling force $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2 =$ **7239.5** kN

Torsion factor $\beta_T = 1 - (y_0 / i_0)^2 =$ **1**

Elastic critical torsional-flexural buckling force $N_{cr,TF} =$ **4180** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **4180** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})} =$ **0.501**

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

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_T =$ **0.34**

Buckling reduction determination factor $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) =$ **0.677**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1) =$ **0.884**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **928.5** kN

$N_{Ed} / N_{b,T,Rd} =$ **0.063**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f =$ **187.6** mm $\eta =$ **1.000**

$h_w / t_w = 29.3 = 31.7 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{y,Ed}$ = **38.7** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1458** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **231.4** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.167**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **65.9** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **86.5** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.762**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.823**
	$C_1 = 1 / k_c^2$ = **1.478**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m3\_s1\_seg1\_B}$ = **1200** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **866.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.316**
Limiting slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.4**
	$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check bending and axial force - Section 6.2.9

Bending and axial force check - eq.6.33 & eq.6.34	$N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **165.1** kN
	$N_{Ed} / N_{y,lim}$ = **0.356**

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	ψ_y = 13.09 kNm / -65.866 kNm = **-0.199**
	α_y = 13.982 kNm / -65.866 kNm = **-0.212**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = -25.433 kNm / -65.866 kNm = **0.386**
	α_{LT} = -44.142 kNm / -65.866 kNm = **0.670**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.736**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **86.5** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **24.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **1050.8** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.418**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.991**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.39**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.826**
	PASS - Combined bending and compression checks are satisfied

Check design 4623 mm along span

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **23.6** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **86.5** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.273**

PASS - Design bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3	ψ_y = 13.09 kNm / -65.866 kNm = **-0.199**
	α_y = 13.982 kNm / -65.866 kNm = **-0.212**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 15.693 kNm / -25.433 kNm = **-0.617**
	α_{LT} = 19.251 kNm / -25.433 kNm = **-0.757**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **86.5** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **24.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **1050.8** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.415**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.977**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.172**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.325**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design 5668 mm along span

Check y-y axis deflection - Section 7.2.1

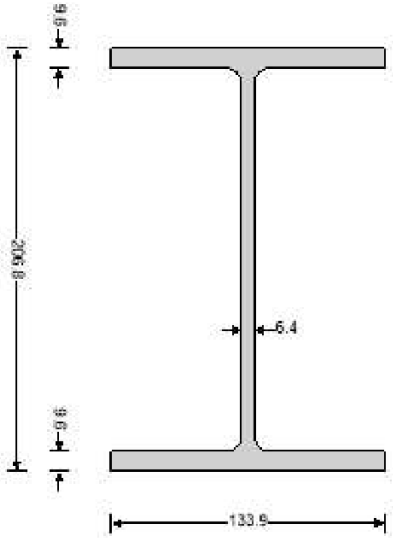
Maximum deflection	δ_y = **28.7** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m3\_s1} / 100$ = **62.1** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.462**

PASS - Allowable deflection exceeds design deflection

4 design

Section details

Section type	UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **9.6** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 205.8 mm
Section breadth, b, 133.9 mm
Flange thickness, t_f, 9.6 mm
Web thickness, t_w, 6.4 mm
Root radius, r, 7.6 mm
Area of section, A, 3821 mm²
Radius of gyration about y-axis, i_y, 87.051 mm
Radius of gyration about z-axis, i_z, 31.728 mm
Elastic section modulus about y-axis, $W_{el,y}$, 280036 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 57454 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 314365 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 88224 mm³
Second moment of area about y-axis, I_y, 28955684 mm⁴
Second moment of area about z-axis, I_z, 3846542 mm⁴

Lateral restraint

Upper flange has full lateral restraint

Lower flange has lateral restraint at supports plus 1200 & 6000 mm

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength)

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} =$ **0.92**

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

Width of section

$c = d =$ **172.4** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.598**

$c / t_w = 26.9 = 29.1 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$c = (b - t_w - 2 \times r) / 2 =$ **56.1** mm

$c / t_f = 5.8 = 6.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force: $N_{Ed} =$ **47** kN

Design resistance of section - eq 6.10: $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **1050.8** kN

$N_{Ed} / N_{c,Rd} =$ **0.045**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length: $L_{cr,y} = L_{m4\_s1} =$ **6210** mm

Critical buckling force: $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **1556.4** kN

Slenderness ratio for buckling - eq 6.50: $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.822**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2: a

Imperfection factor - Table 6.1: $\alpha_y =$ **0.21**

Buckling reduction determination factor: $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.903**

Buckling reduction factor - eq 6.49: $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.783**

Design buckling resistance - eq 6.47: $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **822.9** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.057**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m4\_s1\_seg1\_R}$ = **1200** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **92.7** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **7239.5** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **4180** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.501**

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

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	α_T = **0.34**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.677**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.884**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **928.5** kN
	$N_{Ed} / N_{b,T,Rd}$ = **0.051**

PASS - Design buckling resistance exceeds design compression

Check design 1396 mm along span

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **19.4** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **86.5** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.225**

PASS - Design bending 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	$N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **165.1** kN
	$N_{Ed} / N_{y,lim}$ = **0.301**

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	ψ_y = 12.131 kNm / -67.339 kNm = **-0.180**
	α_y = 8.49 kNm / -67.339 kNm = **-0.126**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 19.282 kNm / -59.945 kNm = **-0.322**
	α_{LT} = 1.235 kNm / -59.945 kNm = **-0.021**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **86.5** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **24.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **1050.8** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.415**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.977**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.154**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.279**

PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web: $h_w = h - 2 \times t_f$ = **187.6** mm η = **1.000**

h_w / t_w = 29.3 = 31.7 × ε / η < 72 × ε / η

Shear buckling resistance can be ignored

Design shear force: $V_{y,Ed}$ = **36** kN

Shear area - cl 6.2.6(3): $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1458** mm²

Design shear resistance - cl 6.2.6(2): $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **231.4** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.156**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment: $M_{y,Ed}$ = **67.3** kNm

Design bending resistance moment - eq 6.13: $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **86.5** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.779**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6: k_c = **0.97**

$C_1 = 1 / k_c^2$ = **1.064**

Poissons ratio: ν = **0.3**

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

Unrestrained effective length: $L = 1.0 \times L_{m4\_s1\_seg3\_B}$ = **210** mm

Elastic critical buckling moment: $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **19063.4** kNm

Slenderness ratio for lateral torsional buckling: $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.067**

Limiting slenderness ratio: $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT}$ < $\bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3: ψ_y = 12.131 kNm / -67.339 kNm = **-0.180**

α_y = 8.49 kNm / -67.339 kNm = **-0.126**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**

ψ_{LT} = -59.945 kNm / -67.339 kNm = **0.890**

α_{LT} = -63.601 kNm / -67.339 kNm = **0.944**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.956**

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

Characteristic moment resistance: $M_{y,Rk} = W_{pl,y} \times f_y$ = **86.5** kNm

Characteristic moment resistance: $M_{z,Rk} = W_{pl,z} \times f_y$ = **24.3** kNm

Characteristic resistance to normal force: $N_{Rk} = A \times f_y$ = **1050.8** kN

Interaction factors: $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.418**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.994**

Interaction formulae - eq 6.61 & eq 6.62: $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.398**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.846**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection — δ_y = **24.1** mm

Allowable deflection — $\delta_{y,Allowable} = L_{m4\_s1} / 100$ = **62.1** mm

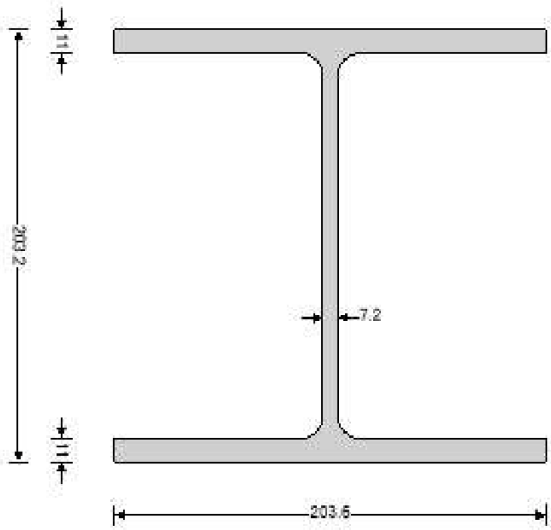
$\delta_y / \delta_{y,Allowable}$ = **0.388**

PASS - Allowable deflection exceeds design deflection

Member5 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **11** mm
Nominal yield strength	f_y = **355** N/mm²
Nominal ultimate tensile strength	f_u = **470** N/mm²
Modulus of elasticity	E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, i_y, 88.19 mm
Radius of gyration about z-axis, i_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230884 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Right flange has full lateral restraint

Left flange has lateral restraint at supports only

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section — c = d = **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.563**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 =$ **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force — $N_{Ed} =$ **50.1** kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **2085** kN

$N_{Ed} / N_{c,Rd} =$ **0.024**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length — $L_{cr,y} = L_{m5\_s1} =$ **2300** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **17896.7** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.341**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — b

Imperfection factor - Table 6.1 — $\alpha_y =$ **0.34**

Buckling reduction determination factor — $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.582**

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.949**

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **1978.1** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.025**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length — $L_{cr,T} = L_{m5\_s1\_seg1\_R} =$ **2300** mm

Distance from shear centre to centroid in y axis — $y_0 =$ **0.0** mm

Distance from shear centre to centroid in z axis — $z_0 =$ **0.0** mm

Radius of gyration — $i_0 = \sqrt{(i_y^2 + i_z^2)} =$ **102.0** mm

Elastic critical torsional buckling force — $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2 =$ **7097.8** kN

Torsion factor — $\beta_T = 1 - (y_0 / i_0)^2 =$ **1**

Elastic critical torsional-flexural buckling force — $N_{cr,TF} =$ **4180** kN

Elastic critical buckling force — $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **4180** kN

Slenderness ratio for torsional buckling - eq 6.52 — $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})} =$ **0.706**

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 — $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) =$ **0.873**

Buckling reduction factor - eq 6.49 — $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1) =$ **0.721**

Design buckling resistance - eq 6.47 — $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **1502.9** kN

$N_{Ed} / N_{b,T,Rd} =$ **0.033**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f =$ **181.2** mm $\eta =$ **1.000**

$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{y,Ed}$ = **48.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.138**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **67.3** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.381**
	PASS - Design bending 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
	$N_{Ed} / N_{y,lim}$ = **0.216**
	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	ψ_y = 43.2 kNm / -67.339 kNm = **-0.642**
	α_y = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4)$ = **0.400**
	ψ_{LT} = 43.2 kNm / -67.339 kNm = **-0.642**
	α_{LT} = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** 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**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.988**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.178**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.407**
	PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm $\quad \eta$ = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **48.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.138**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **43.2** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm

$M_{y,Ed}$ / $M_{c,y,Rd}$ = **0.245**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.65**
	$C_1 = 1 / k_c^2$ = **2.367**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m5\_s1\_seg1\_T}$ = **2300** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **1584.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.334**
Limiting slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3	ψ_y = 43.2 kNm / -67.339 kNm = **-0.642**
	α_y = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4)$ = **0.400**
	ψ_{LT} = 43.2 kNm / -67.339 kNm = **-0.642**
	α_{LT} = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl.y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl.z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** 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**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.988**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.124**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.273**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection	δ_y = **15.2** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m5\_s1} / 100$ = **23** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.662**

PASS - Allowable deflection exceeds design deflection

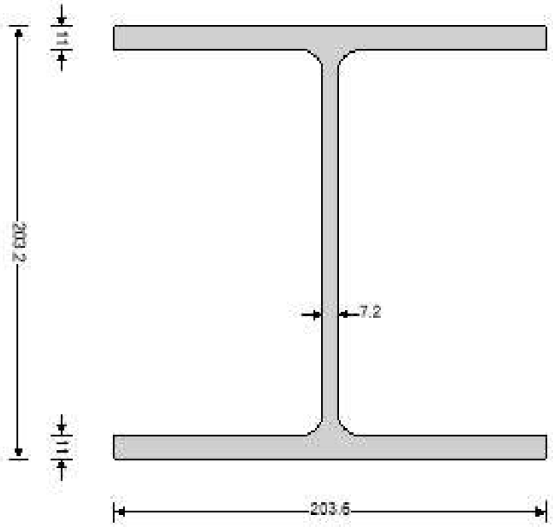
Member6 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **11** mm
Nominal yield strength	f_y = **355** N/mm²

Nominal ultimate tensile strength f_u = **470** N/mm²

Modulus of elasticity E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, i_y, 88.19 mm
Radius of gyration about z-axis, i_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230864 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section c = d = **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.569**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force N_{Ed} = **54.6** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = A \times f_y / \gamma_{M0}$ = **2085** kN

$N_{Ed} / N_{c,Rd}$ = **0.026**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length $L_{cr,y} = L_{m6\_s1}$ = **3400** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **8189.7** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.505**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 α_y = **0.34**

Buckling reduction determination factor $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.679**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.882**

Design buckling resistance - eq 6.47	$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **1839.3** kN
	$N_{Ed} / N_{b,y,Rd}$ = **0.03**
	PASS - Design buckling resistance exceeds design compression

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

Critical buckling length	$L_{cr,z} = L_{m6\_s1\_seg1}$ = **3400** mm
Critical buckling force	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **2775.8** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **0.867**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_z = **0.49**
Buckling reduction determination factor	$\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **1.039**
Buckling reduction factor - eq 6.49	$\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.62**
Design buckling resistance - eq 6.47	$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **1293.6** kN
	$N_{Ed} / N_{b,z,Rd}$ = **0.042**
	PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m6\_s1\_seg1\_R}$ = **3400** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **102.0** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **4180** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **4180** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.706**

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

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_T = **0.49**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.873**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.721**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **1502.9** kN
	$N_{Ed} / N_{b,T,Rd}$ = **0.036**
	PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm $\quad \eta$ = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **13.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.038**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed}$ = **31.2** kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.177**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.676**

$C_1 = 1 / k_c^2$ = **2.19**

Poissons ratio — ν = **0.3**

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

Unrestrained effective length — $L = 1.0 \times L_{m6\_s1\_seg1\_T}$ = **3400** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **761.3** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.482**

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 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.601**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.968**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.871**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**

Design buckling resistance moment - eq 6.55 — $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.177**

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

$N_{Ed} / N_{y,lim}$ = **0.236**

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 — ψ_y = -13.355 kNm / 31.171 kNm = **-0.428**

α_y = 8.908 kNm / 31.171 kNm = **0.286**

$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4)$ = **0.429**

ψ_{LT} = -13.355 kNm / 31.171 kNm = **-0.428**

α_{LT} = 8.908 kNm / 31.171 kNm = **0.286**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4)$ = **0.429**

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

Characteristic moment resistance — $M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm

Characteristic moment resistance — $M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm

Characteristic resistance to normal force — $N_{Rk} = A \times f_y$ = **2085** kN

Interaction factors — $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.432**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.980**

Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.109**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.221**
	PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm η = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **13.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.038**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **13.4** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.076**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.676**
	$C_1 = 1 / k_c^2$ = **2.19**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m6\_s1\_seg1\_B}$ = **3400** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **761.3** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.482**
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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.601**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.968**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.871**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.076**
	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	ψ_y = -13.355 kNm / 31.171 kNm = **-0.428**
	α_y = 8.908 kNm / 31.171 kNm = **0.286**

C_{my} = max(0.6 + 0.4 × ψ_y, 0.4) = **0.429**

ψ_{LT} = -13.355 kNm / 31.171 kNm = **-0.428**

α_{LT} = 8.908 kNm / 31.171 kNm = **0.286**

C_{mLT} = max(0.6 + 0.4 × ψ_{LT}, 0.4) = **0.429**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.433**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.979**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.065**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.12**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection	δ_y = **5.1** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m6\_s1} / 100$ = **34** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.149**

PASS - Allowable deflection exceeds design deflection

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Our ref: 07229E_sH_SE_RP_002

Dr Sally Murrall-Smith PhD
Totnes Renewable Energy Society
107 High Street
Totnes
TQ9 5SN

21st January 2026

Dear Sally,

Structural Check at Dart Marine Park, Steamer Quay Road, Totnes, TQ9 5AL – Building B

As requested, and commissioned, we have carried out a structural survey of the portal frame building at Dart Marine Park, carried out calculations and prepared this report, for the purpose of justifying the proposed PV panel installation to the roof.

The building is of a two storey portalised steel frame, we could not measure the floor steel due to the finishes of the building, we requested 'As-Built' drawings, but these haven't been available. However, we are confident that the compressive utilisation of the steel columns is not at question, more the overall stability, as such we have included a tie beam at floor level but not analysed this element.

The portal steel frame has 203 x 133 x 30 UB Rafters and 203 UC 46 Columns, the portal frames are at 5.7 metre centres. The dimension of the portal frame is shown in the calculations within this report.

An element of the building is cantilevering over the river path, due to the finishes of the building we could not measure the steels in this area, we requested 'As-Built' drawings, but these haven't been available. Therefore, we can not justify this area on the building.

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VAT Registration Nº GB 190 6091 09

The proposed PV system is fixed to the roof, loadings are taken as 0.15kN/m^2 which is applied on top of the snow loading at 0.6kN/m^2 and the dead weight loading of 0.5kN/m^2 including services.

Structural calculations have been carried out to check the capacity of these members, and determine adequacy of the existing structure for the proposed PV panel instillation.

We are pleased to confirm that the steel frame member sizes are adequate to support the additional vertical loading of the proposed installation.

Therefore, we can confirm that installing the new roof panels upon the existing roof of this structure will not have a structurally detrimental effect.

EXCEPT THE CANTILEVER AREA AS BELOW: -



Red box shown above in the no PV zone

This letter confirms cover under our PI should any failure of the steel frame occur due to inadequate capacity.

This letter is not intended to and does not relate to the adequacy or assessment of the fixing system to the Photovoltaics to the roof.

Yours sincerely,



Mark Harris MIStructE
Director
structureHaus Ltd

Appendix A – Information Provided and Photos

Appendix B – Structural Calculations

APPENDIX A – INFORMATION PROVIDED & SITE PHOTOS



Proposed PV layout



Site Visit Photo



Site Visit Photo

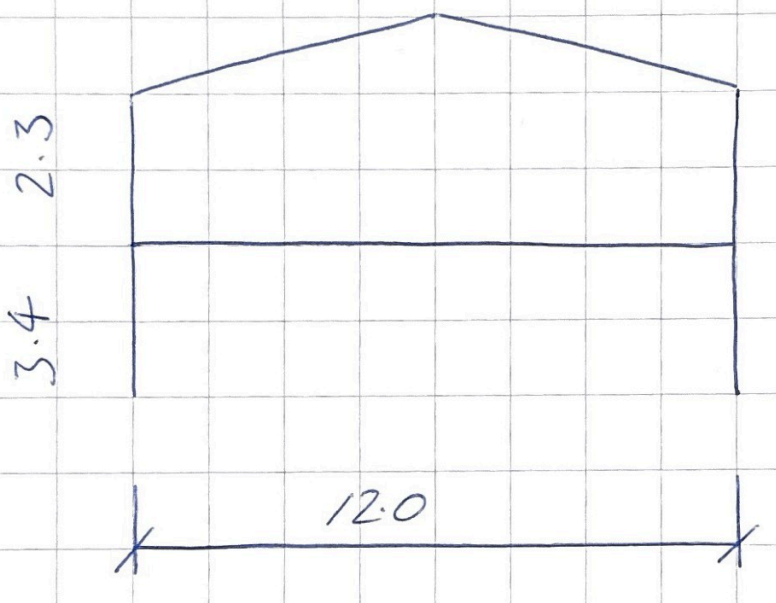
APPENDIX B – STRUCTURAL CALCULATIONS

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Ref 07229E
Date Jan 26
Sheet 1
By MDH

Calculation Contract Dart Marine Park

Building B



@ 5.7 m c/c's

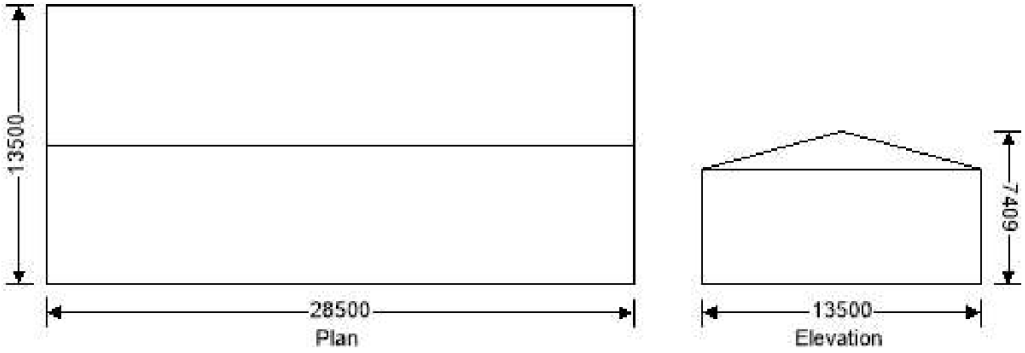
Rafters 203 × 133 × 30 UB

Columns 203 UC 46

WIND LOADING (BS6399)

In accordance with BS6399

Tedds calculation version 3.0.19



Building data

Type of roof	Duopitch
Length of building	L = **28500** mm
Width of building	W = **13500** mm
Height to eaves	H = **5600** mm
Pitch of roof	α_0 = **15.0** deg
Reference height	H_r = **7409** mm

Dynamic classification

Building type factor (Table 1)	K_b = **1.0**
Dynamic augmentation factor (1.6.1)	$C_r = [K_b \times (H_r / (0.1\ m))^{0.75}] / (800 \times \log(H_r / (0.1\ m))) =$ **0.02**

Site wind speed

Location	Totnes
Basic wind speed (Figure 6 BS6399:Pt 2)	V_b = **22.5** m/s
Site altitude	Δs = **1** m
Upwind distance from sea to site	d_{sea} = **10** km
Direction factor	S_d =**0.82**
Seasonal factor	S_s = **1.00**
Probability factor	S_p = **1.00**
Critical gap between buildings	g = **5000** mm
Topography not significant	
Altitude factor	$S_a = 1 + 0.001 \times \Delta s$ / 1m = **1.00**
Site wind speed	$V_s = V_b \times S_a \times S_d \times S_s \times S_p$ = **18.5** m/s
Terrain category	Sea
Displacement height (sheltering effect excluded)	H_d = 0mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

Dynamic pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought)	H_{ref} = **5600**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **5600**mm
Fetch factor (Table 22)	S_c = **0.959**

Turbulence factor (Table 22)	S_t = **0.190**
Gust peak factor	g_t = **3.44**
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.59**
Effective wind speed	$V_e = V_s \times S_b$ = **29.3** m/s
Dynamic pressure	q_s = 0.613 kg/m^3 × V_e^2 = **0.527** kN/m^2

Dynamic pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought)	H_{ref} = **7409**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **7409**mm
Fetch factor (Table 22)	S_c = **1.005**
Turbulence factor (Table 22)	S_t = **0.185**
Gust peak factor	g_t = **3.44**
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.64**
Effective wind speed	$V_e = V_s \times S_b$ = **30.4** m/s
Dynamic pressure	q_s = 0.613 kg/m^3 × V_e^2 = **0.566** kN/m^2

Size effect factors

Diagonal dimension for gablewall	a_{eg} = **14.6** m
External size effect factor gablewall	C_{aeg} = **0.903**
Diagonal dimension for side wall	a_{es} = **29.0** m
External size effect factor side wall	C_{aes} = **0.841**
Diagonal dimension for roof	a_{er} = **29.3** m
External size effect factor roof	C_{aer} = **0.840**
Room/storey volume for internal size effect factor	V_i = **1740.000** m^3
Diagonal dimension for internal size effect factors	$a_i = 10 \times (V_i)^{1/3}$ = **120.277** m
Internal size effect factor	C_{ai} = **0.712**

Pressures and forces

Net pressure	$p = q_s \times c_{pe} \times C_{ae} - q_s \times c_{pi} \times C_{ai}$
Net force	$F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.10	0.57	0.840	-0.60	22.73	-13.71
B (-ve)	-0.80	0.57	0.840	-0.46	20.99	-9.67
C (-ve)	-0.40	0.57	0.840	-0.27	155.44	-42.06
E (-ve)	-1.30	0.57	0.840	-0.70	22.73	-15.87
F (-ve)	-0.90	0.57	0.840	-0.51	20.99	-10.66
G (-ve)	-0.50	0.57	0.840	-0.32	155.44	-49.44

Total vertical net force	$F_{w,v}$ = **-136.59** kN
Total horizontal net force	$F_{w,h}$ = **2.73** kN

Walls load case 1 - Wind 0, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.40	0.57	0.903	-0.80	17.77	-14.19
B	-0.83	0.57	0.903	-0.51	70.04	-35.51
w	0.73	0.53	0.841	0.25	159.60	39.78
l	-0.50	0.53	0.841	-0.30	159.60	-47.29

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-47.3** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **39.8** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **77.6** kN

Roof load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A (+ve)	0.20	0.57	0.840	0.22	22.73	4.91
B (+ve)	0.20	0.57	0.840	0.22	20.99	4.53
C (+ve)	0.20	0.57	0.840	0.22	155.44	33.55
E (+ve)	-1.30	0.57	0.840	-0.50	22.73	-11.29
F (+ve)	-0.90	0.57	0.840	-0.31	20.99	-6.44
G (+ve)	-0.50	0.57	0.840	-0.12	155.44	-18.14

Total vertical net force $F_{w,v}$ = **6.87** kN

Total horizontal net force $F_{w,h}$ = **20.41** kN

Walls load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.40	0.57	0.903	-0.60	17.77	-10.61
B	-0.83	0.57	0.903	-0.31	70.04	-21.41
w	0.73	0.53	0.841	0.44	159.60	69.70
l	-0.50	0.53	0.841	-0.11	159.60	-17.38

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-17.4** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **69.7** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **92.9** kN

Roof load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.60	0.57	0.840	-0.84	9.43	-7.93
B (-ve)	-1.50	0.57	0.840	-0.79	9.43	-7.48
C (-ve)	-0.60	0.57	0.840	-0.37	75.47	-27.59
D (-ve)	-0.40	0.57	0.840	-0.27	303.98	-82.25

Total vertical net force $F_{w,v}$ = **-120.99** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.54	0.53	0.841	-0.75	12.54	-9.47
B	-0.88	0.53	0.841	-0.46	50.18	-23.28
C	-0.81	0.53	0.841	-0.44	96.88	-42.19
w	0.61	0.57	0.903	0.23	87.81	20.41
l	-0.50	0.57	0.903	-0.34	87.81	-29.50

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-29.5** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **20.4** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **43.1** kN

Roof load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.60	0.57	0.840	-0.64	9.43	-6.03
B (-ve)	-1.50	0.57	0.840	-0.59	9.43	-5.58
C (-ve)	-0.60	0.57	0.840	-0.16	75.47	-12.39
D (-ve)	-0.40	0.57	0.840	-0.07	303.98	-21.04

Total vertical net force $F_{w,v}$ = **-43.51** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

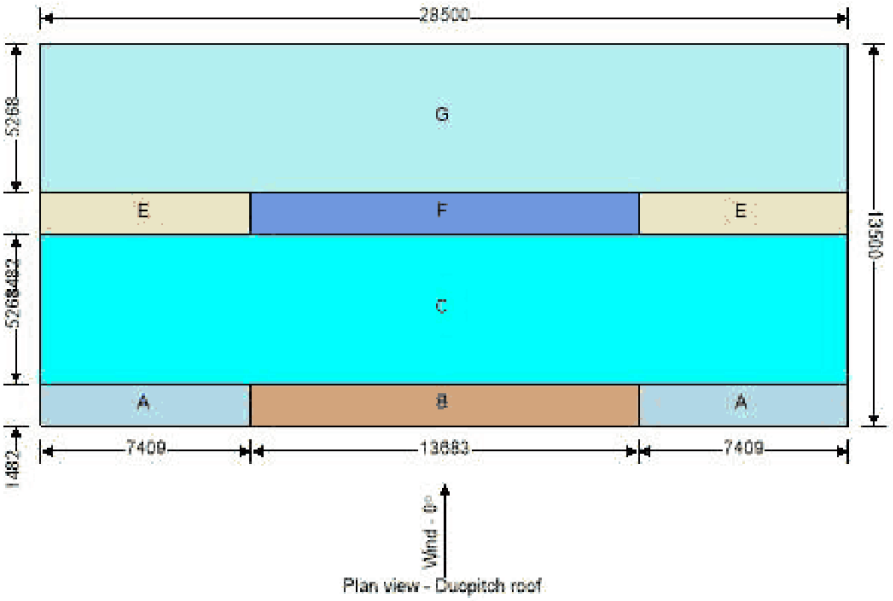
Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.54	0.53	0.841	-0.57	12.54	-7.12
B	-0.88	0.53	0.841	-0.28	50.18	-13.87
C	-0.81	0.53	0.841	-0.25	96.88	-24.03
w	0.61	0.57	0.903	0.43	87.81	38.09
l	-0.50	0.57	0.903	-0.13	87.81	-11.82

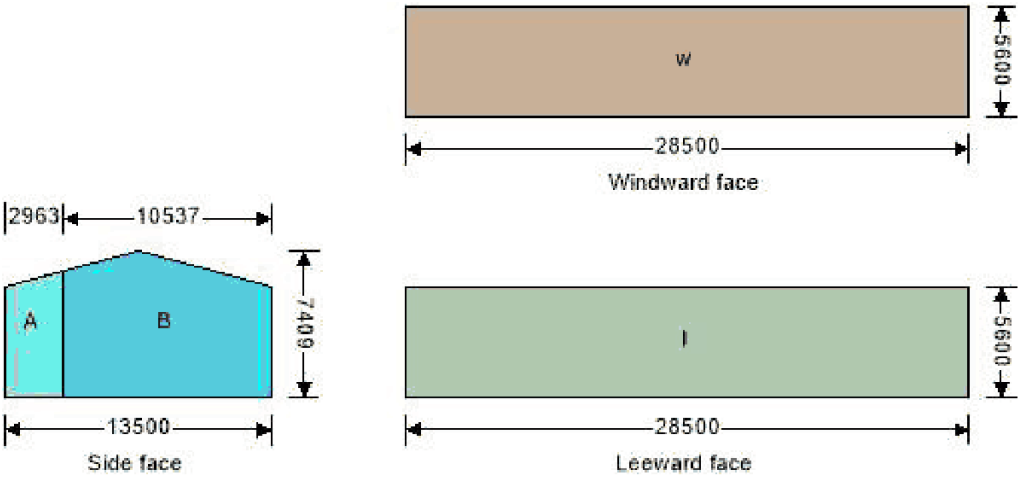
Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-11.8** kN

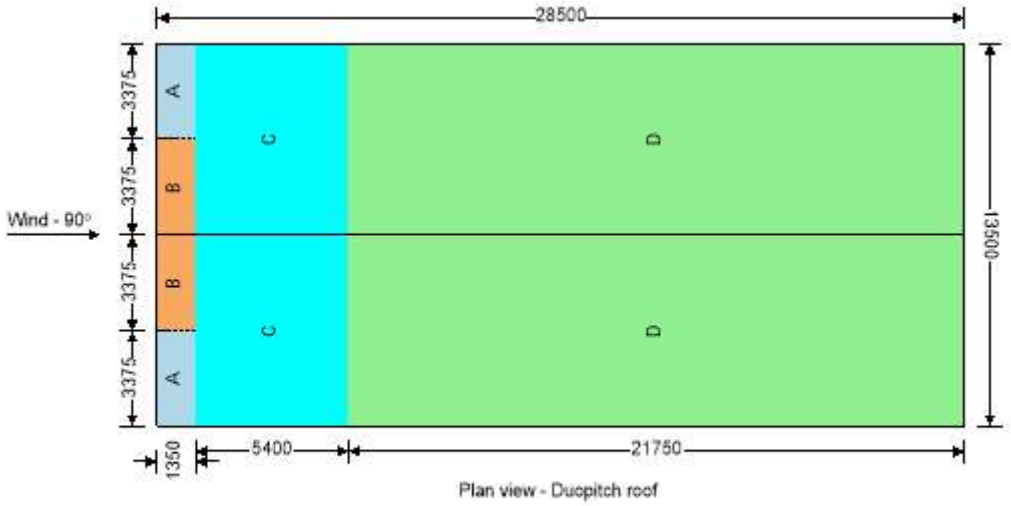
Net windward force for overall section $F_w = F_{w,ww}$ = **38.1** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **43.1** kN

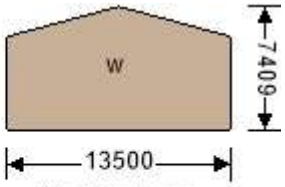




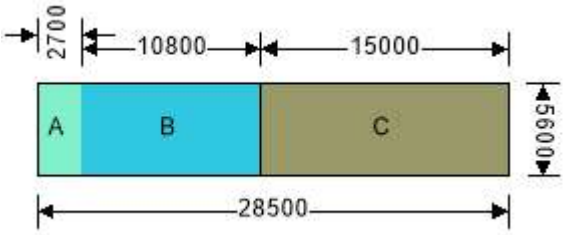
structureHaus Ltd	Project: Dart Marine Park, Totnes				Job no.: 07229E	
	Calcs for: Wind				Start page no./Revision: 7	
	Calcs by: MDH	Calcs date: 11/11/2025	Checked by: DALP	Checked date:	Approved by:	Approved date:



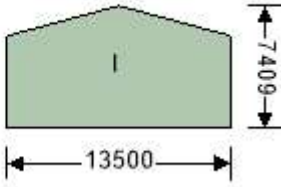
Plan view - Duopitch roof



Windward face

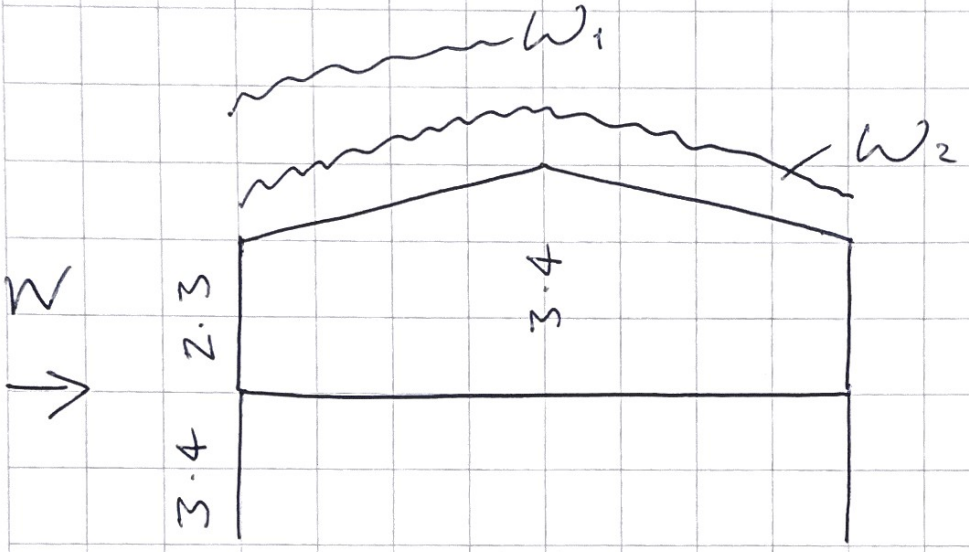


Side face



Leeward face

Building B



@ 5.7m c/c's

$W = 1.12 \times 5.7 = 6.38\ kN/m$

$W_1 = 0.12 \times 5.7 = 0.68\ kN/m$

$W_2 = 0.5 \times 5.7 = 2.85\ kN/m$

$0.6 = 3.42\ kN/m$

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

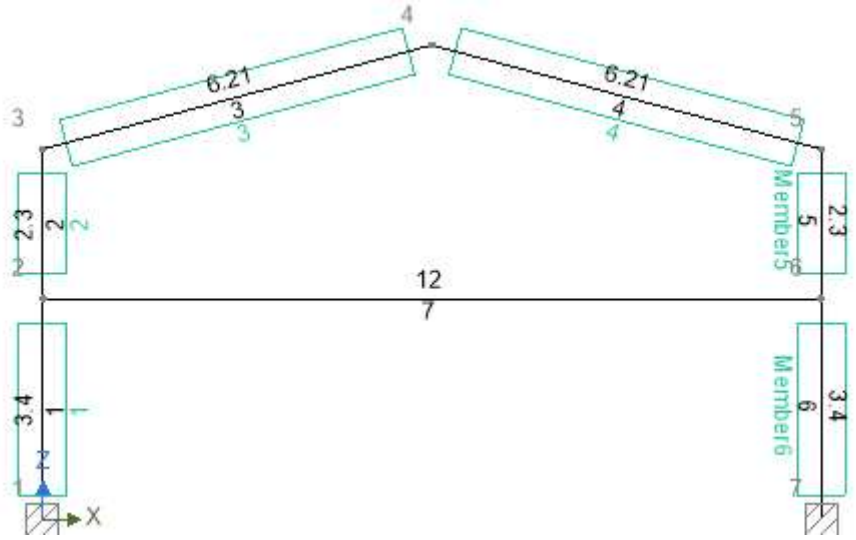
Tedds calculation version 4.5.06

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (m) - Steel (EC3)



Materials

Name	Density (kg/m³)	Youngs Modulus kN/mm²	Shear Modulus kN/mm²	Thermal Coefficient °C⁻¹
Steel (EC3)	7850	210	80.8	0.000012

Sections

Name	Area (cm²)	Moment of inertia Major (cm⁴)	Moment of inertia Minor (cm⁴)	Shear area parallel to Minor (cm²)	Shear area parallel to Major (cm²)
UKC 203x203x46	58.7	4567.8	1548.2	14.6	40.3
UB 203x133x30	38.2	2895.6	384.7	13.2	23.1

Nodes

Node	Co-ordinates X (m)	Co-ordinates Z (m)	Freedom X	Freedom Z	Freedom Rot.	Coordinate system Name	Coordinate system Angle (°)	Spring X (kN/m)	Spring Z (kN/m)	Spring Rot. kNm/°
1	0	0	Fixed	Fixed	Fixed		0	0	0	0
2	0	3.4	Free	Free	Free		0	0	0	0
3	0	5.7	Free	Free	Free		0	0	0	0
4	6	7.3	Free	Free	Free		0	0	0	0
5	12	5.7	Free	Free	Free		0	0	0	0
6	12	3.4	Free	Free	Free		0	0	0	0
7	12	0	Fixed	Fixed	Fixed		0	0	0	0

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	3.4	1	2	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
2	2.3	2	3	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
3	6.21	3	4	UB 203x133x30	Steel (EC3)	Fixed	Fixed	Fixed	
4	6.21	4	5	UB 203x133x30	Steel (EC3)	Fixed	Fixed	Fixed	
5	2.3	5	6	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
6	3.4	6	7	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
7	12	2	6	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	

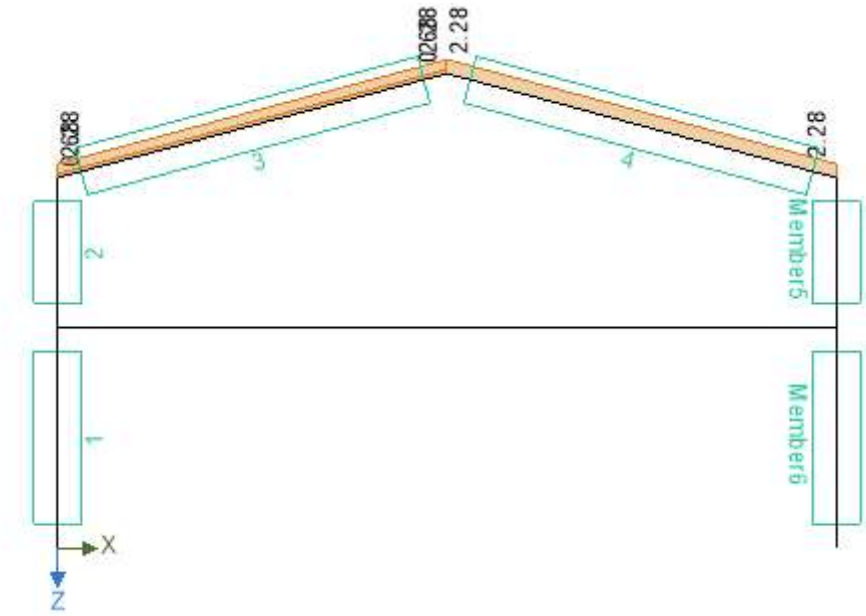
Members

Name	Elements	
	Start	End
1	1	1
2	2	2
3	3	3
4	4	4
Member5	5	5
Member6	6	6

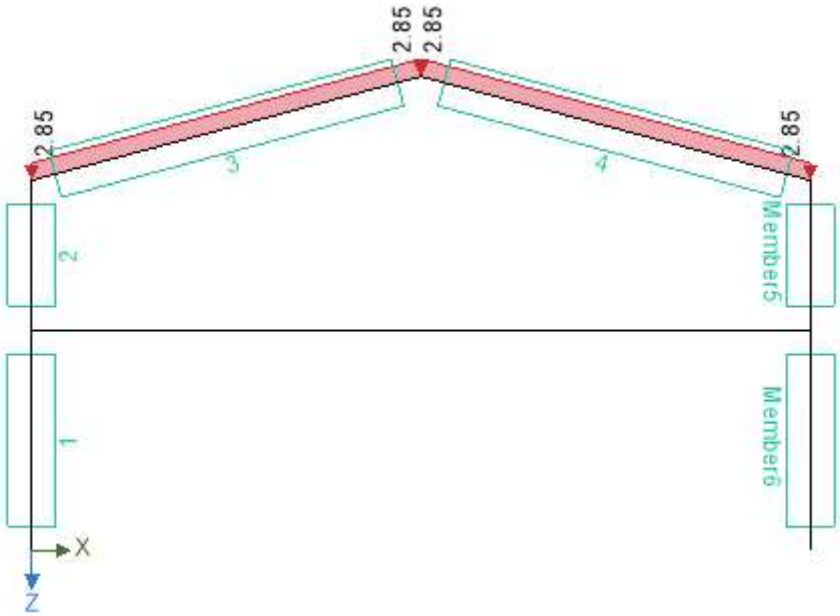
Loading

Self weight included

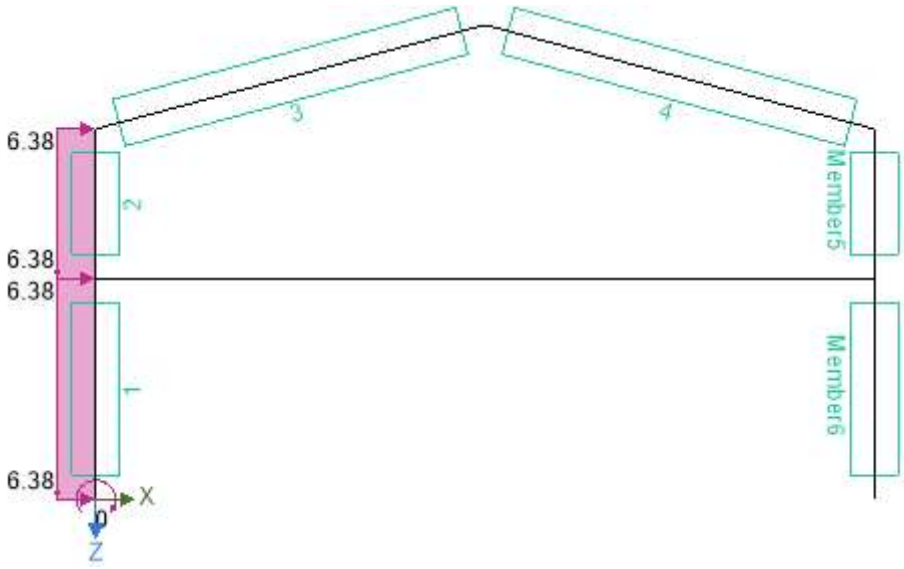
Permanent - Loading (kN/m)



Imposed - Loading (kN/m)



Wind - Loading (kN/m,kNm)



Load combination factors

Load combination	Self Weight	Permanent	Imposed	Wind
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50	
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00	
1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	1.50	0.75
1.0G + 1.0Q + 0.5S + 0.5W (Service)	1.00	1.00	1.00	0.50

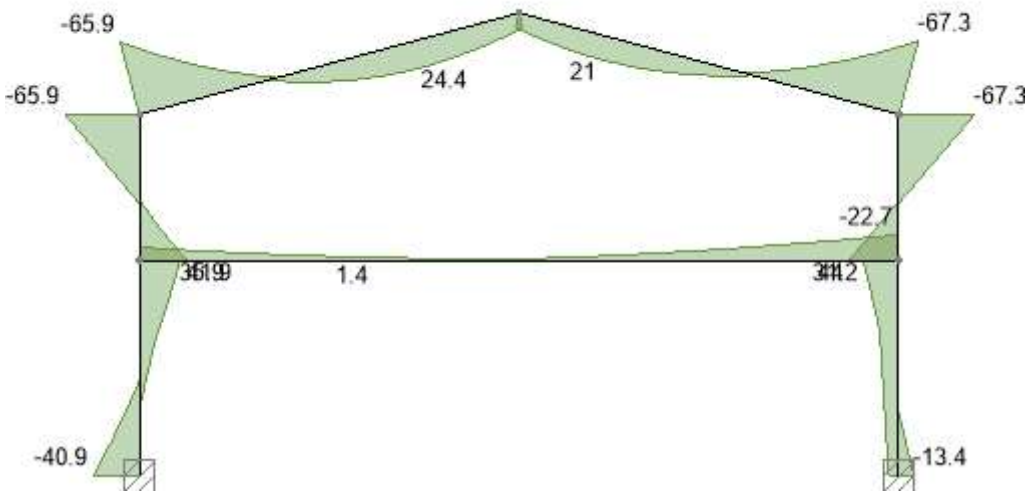
Member Loads

Member	Load case	Load Type	Orientation	Description
3	Permanent	UDL	GlobalZ	0.68 kN/m
3	Permanent	UDL	GlobalZ	2.28 kN/m
4	Permanent	UDL	GlobalZ	2.28 kN/m
3	Imposed	UDL	GlobalZ	2.85 kN/m
4	Imposed	UDL	GlobalZ	2.85 kN/m
1	Wind	Point Moment	GlobalY	0 kNm at 0 m
1	Wind	UDL	GlobalX	6.38 kN/m
2	Wind	UDL	GlobalX	6.38 kN/m

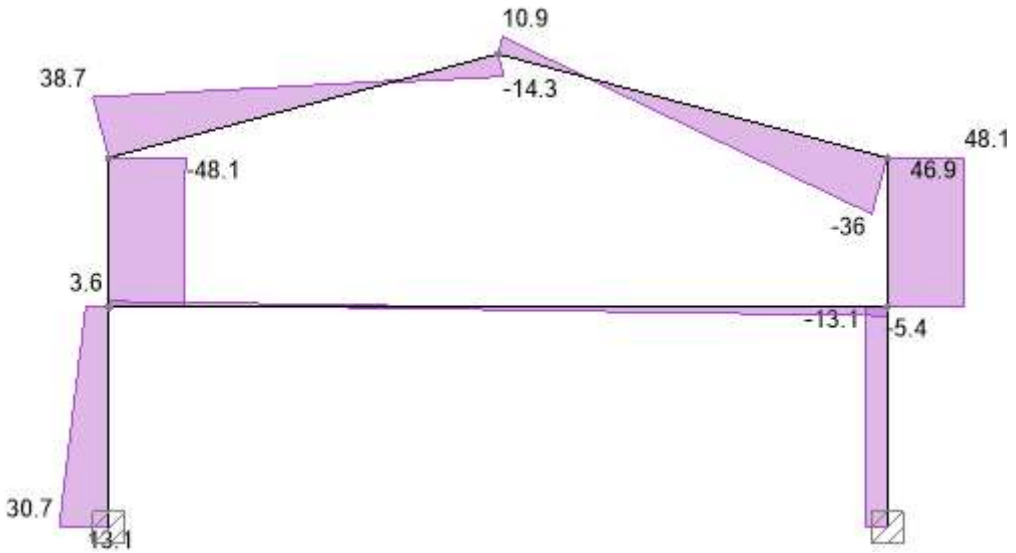
Results

Forces

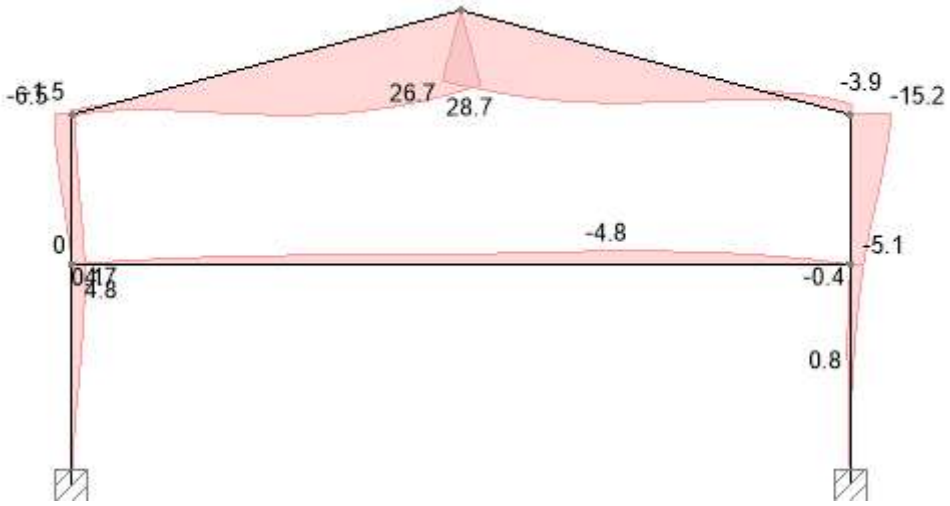
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



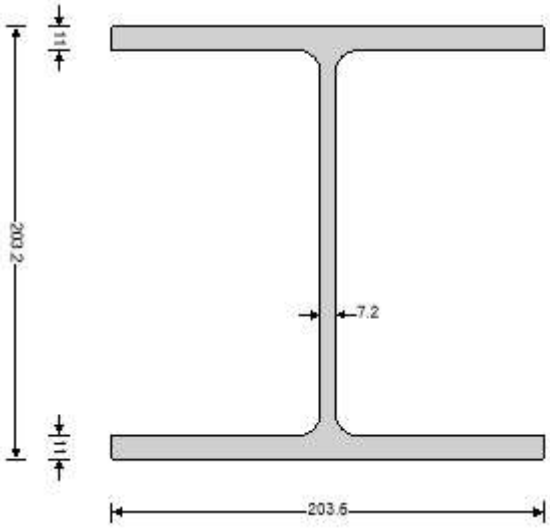
Partial factors - Section 6.1

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**

1 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	t_{nom} = max(t_f, t_w) = **11** mm
Nominal yield strength	f_y = **355** N/mm²
Nominal ultimate tensile strength	f_u = **470** N/mm²
Modulus of elasticity	E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, i_y, 88.19 mm
Radius of gyration about z-axis, i_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230864 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section

$c = d =$ **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.570**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$c = (b - t_w - 2 \times r) / 2 =$ **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force — $N_{Ed} =$ **57.2** kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **2085** kN

$N_{Ed} / N_{c,Rd} =$ **0.027**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length — $L_{cr,y} = L_{m1\_s1} =$ **3400** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **8189.7** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.505**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — b

Imperfection factor - Table 6.1 — $\alpha_y =$ **0.34**

Buckling reduction determination factor — $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.679**

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.882**

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **1839.3** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.031**

PASS - Design buckling resistance exceeds design compression

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

Critical buckling length — $L_{cr,z} = L_{m1\_s1\_seg1} =$ **3400** mm

Critical buckling force — $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **2775.8** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **0.867**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — c

Imperfection factor - Table 6.1 — $\alpha_z =$ **0.49**

Buckling reduction determination factor — $\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **1.039**

Buckling reduction factor - eq 6.49 — $\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1) =$ **0.62**

Design buckling resistance - eq 6.47 — $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **1293.6** kN

$N_{Ed} / N_{b,z,Rd} =$ **0.044**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m1\_s1\_seg1\_R}$ = **3400** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **102.0** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **4180** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **4180** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.706**

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

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_T = **0.49**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.873**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.721**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **1502.9** kN
	$N_{Ed} / N_{b,T,Rd}$ = **0.038**
	PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm η = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **30.7** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.088**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **40.9** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.231**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.634**
	$C_1 = 1 / k_c^2$ = **2.49**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m1\_s1\_seg1\_B}$ = **3400** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **865.6** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.452**
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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.585**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.980**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.861**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.231**

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
	$N_{Ed} / N_{y,lim}$ = **0.247**

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	ψ_y = 35.916 kNm / -40.856 kNm = **-0.879**
	α_y = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 35.916 kNm / -40.856 kNm = **-0.879**
	α_{LT} = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.404**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.974**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.126**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.274**

PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm η = **1.000**
	h_w / t_w = 25.2 = 30.9 × ε / η < 72 × ε / η

Shear buckling resistance can be ignored

Design shear force	$V_{y,Ed}$ = **14.4** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm^2
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.042**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **35.9** kNm

Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.203**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.634**
	$C_1 = 1 / k_c^2$ = **2.49**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2
Unrestrained effective length	$L = 1.0 \times L_{m1\_s1\_seg1\_T}$ = **3400** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **865.6** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.452**
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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.585**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.980**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.861**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.203**
	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	ψ_y = 35.916 kNm / -40.856 kNm = **-0.879**
	α_y = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 35.916 kNm / -40.856 kNm = **-0.879**
	α_{LT} = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.404**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.975**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.114**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.245**
	PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design 3199 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection — δ_y = **4.8** mm

Allowable deflection — $\delta_{y,Allowable} = L_{m1\_s1} / 100$ = **34** mm

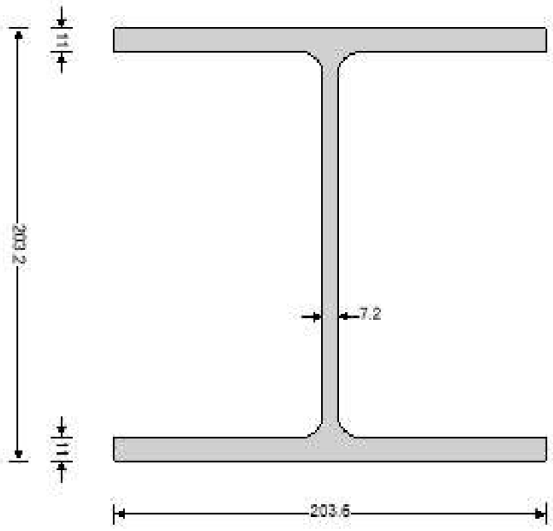
$\delta_y / \delta_{y,Allowable}$ = **0.14**

PASS - Allowable deflection exceeds design deflection

2 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **11** mm
Nominal yield strength	f_y = **355** N/mm²
Nominal ultimate tensile strength	f_u = **470** N/mm²
Modulus of elasticity	E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, i_y, 88.19 mm
Radius of gyration about z-axis, i_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230884 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section — c = d = **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.566**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force	N_{Ed} = **54** kN
Design resistance of section - eq 6.10	$N_{c,Rd} = A \times f_y / \gamma_{M0}$ = **2085** kN
	$N_{Ed} / N_{c,Rd}$ = **0.026**
	PASS - Design compression resistance exceeds design compression

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

Critical buckling length	$L_{cr,y} = L_{m2\_s1}$ = **2300** mm
Critical buckling force	$N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **17896.7** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.341**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	α_y = **0.34**
Buckling reduction determination factor	$\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.582**
Buckling reduction factor - eq 6.49	$\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.949**
Design buckling resistance - eq 6.47	$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **1978.1** kN
	$N_{Ed} / N_{b,y,Rd}$ = **0.027**
	PASS - Design buckling resistance exceeds design compression

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

Critical buckling length	$L_{cr,z} = L_{m2\_s1\_seg1}$ = **2300** mm
Critical buckling force	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **6065.9** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **0.586**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_z = **0.49**
Buckling reduction determination factor	$\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **0.766**
Buckling reduction factor - eq 6.49	$\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.793**
Design buckling resistance - eq 6.47	$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **1654.4** kN
	$N_{Ed} / N_{b,z,Rd}$ = **0.033**
	PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m2\_s1\_seg1\_R}$ = **2300** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **102.0** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **7097.8** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **4180** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.706**

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

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_T = **0.49**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.873**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.721**

Design buckling resistance - eq 6.47

$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **1502.9** kN

$N_{Ed} / N_{b,T,Rd} =$ **0.036**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web

$h_w = h - 2 \times t_f =$ **181.2** mm $\eta =$ **1.000**

$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force

$V_{y,Ed} =$ **46.9** kN

Shear area - cl 6.2.6(3)

$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1698** mm²

Design shear resistance - cl 6.2.6(2)

$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **347.9** kN

$V_{y,Ed} / V_{c,y,Rd} =$ **0.135**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$M_{y,Ed} =$ **41.9** kNm

Design bending resistance moment - eq 6.13

$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **176.6** kNm

$M_{y,Ed} / M_{c,y,Rd} =$ **0.237**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$k_c =$ **0.65**

$C_1 = 1 / k_c^2 =$ **2.367**

Poissons ratio

$\nu =$ **0.3**

Shear modulus

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

Unrestrained effective length

$L = 1.0 \times L_{m2\_s1\_seg1\_T} =$ **2300** mm

Elastic critical buckling moment

$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **1584.8** kNm

Slenderness ratio for lateral torsional buckling

$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} =$ **0.334**

Limiting slenderness ratio

$\bar{\lambda}_{LT,0} =$ **0.4**

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

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

$N_{Ed} / N_{y,lim} =$ **0.233**

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

$\psi_y = 41.933$ kNm / -65.866 kNm = **-0.637**

$\alpha_y = -11.966$ kNm / -65.866 kNm = **0.182**

$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4) =$ **0.400**

$\psi_{LT} = 41.933$ kNm / -65.866 kNm = **-0.637**

$\alpha_{LT} = -11.966$ kNm / -65.866 kNm = **0.182**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance

$M_{y,Rk} = W_{pl,y} \times f_y =$ **176.6** kNm

Characteristic moment resistance

$M_{z,Rk} = W_{pl,z} \times f_y =$ **82** kNm

Characteristic resistance to normal force

$N_{Rk} = A \times f_y =$ **2085** kN

Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.402**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.987**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.123**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.267**
	PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f =$ **181.2** mm $\eta =$ **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed} =$ **46.9** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1698** mm^2
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd} =$ **0.135**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed} =$ **65.9** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd} =$ **0.373**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	$k_c =$ **0.65**
	$C_1 = 1 / k_c^2 =$ **2.367**
Poissons ratio	$\nu =$ **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)] =$ **80769** N/mm^2
Unrestrained effective length	$L = 1.0 \times L_{m2\_s1\_seg1\_B} =$ **2300** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **1584.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} =$ **0.334**
Limiting slenderness ratio	$\bar{\lambda}_{LT,0} =$ **0.4**
	$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3	$\psi_y =$ 41.933 kNm / -65.866 kNm = **-0.637**
	$\alpha_y =$ -11.966 kNm / -65.866 kNm = **0.182**
	$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4) =$ **0.400**
	$\psi_{LT} =$ 41.933 kNm / -65.866 kNm = **-0.637**
	$\alpha_{LT} =$ -11.966 kNm / -65.866 kNm = **0.182**
	$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y =$ **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y =$ **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y =$ **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.402**

k_{zy} = 1 - 0.1 × min(1, $\bar{\lambda}_z$) × N_{Ed} / ((C$_{mLT}$ - 0.25) × χ_z × N_{Rk} / γ_{M1}) = **0.988**

Interaction formulae - eq 6.61 & eq 6.62

N_{Ed} / (χ_y × N_{Rk} / γ_{M1}) + k_{yy} × $M_{y,Ed}$ / (χ_{LT} × $M_{y,Rk}$ / γ_{M1}) = **0.176**

N_{Ed} / (χ_z × N_{Rk} / γ_{M1}) + k_{zy} × $M_{y,Ed}$ / (χ_{LT} × $M_{y,Rk}$ / γ_{M1}) = **0.4**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection: δ_y = **4.7** mm

Allowable deflection: $\delta_{y,Allowable}$ = L_{m2_s1} / 100 = **23** mm

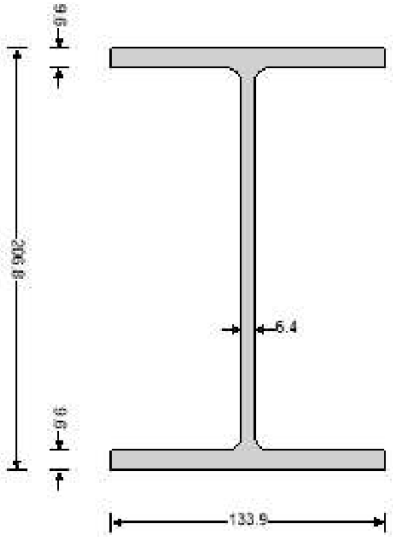
δ_y / $\delta_{y,Allowable}$ = **0.205**

PASS - Allowable deflection exceeds design deflection

3 design

Section details

Section type	UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	t_{nom} = max(t_f, t_w) = **9.6** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 206.8 mm
Section breadth, b, 133.9 mm
Flange thickness, t_f, 9.6 mm
Web thickness, t_w, 6.4 mm
Root radius, r, 7.6 mm
Area of section, A, 3821 mm^2
Radius of gyration about y-axis, i_y, 87.051 mm
Radius of gyration about z-axis, i_z, 31.728 mm
Elastic section modulus about y-axis, $W_{el,y}$, 280036 mm^3
Elastic section modulus about z-axis, $W_{el,z}$, 57454 mm^3
Plastic section modulus about y-axis, $W_{pl,y}$, 314366 mm^3
Plastic section modulus about z-axis, $W_{pl,z}$, 88224 mm^3
Second moment of area about y-axis, I_y, 28955684 mm^4
Second moment of area about z-axis, I_z, 3846542 mm^4

Lateral restraint

Upper flange has full lateral restraint

Lower flange has lateral restraint at supports plus 1200 & 6000 mm

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

ε = √[235 N/mm^2 / f_y] = **0.92**

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

Width of section: c = d = **172.4** mm

α = min([h / 2 + N_{Ed} / (2 × t_w × f_y) - (t_f + r)] / c, 1) = **0.597**

$c / t_w = 26.9 = 29.1 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **56.1** mm

$c / t_f = 5.8 = 6.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force $N_{Ed} =$ **58.8** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **1050.8** kN

$N_{Ed} / N_{c,Rd} =$ **0.056**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length $L_{cr,y} = L_{m3\_s1} =$ **6210** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **1556.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.822**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_y =$ **0.21**

Buckling reduction determination factor $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.903**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.783**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **822.9** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.071**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length $L_{cr,T} = L_{m3\_s1\_seg1\_R} =$ **1200** mm

Distance from shear centre to centroid in y axis $y_0 =$ **0.0** mm

Distance from shear centre to centroid in z axis $z_0 =$ **0.0** mm

Radius of gyration $i_0 = \sqrt{(i_y^2 + i_z^2)} =$ **92.7** mm

Elastic critical torsional buckling force $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2 =$ **7239.5** kN

Torsion factor $\beta_T = 1 - (y_0 / i_0)^2 =$ **1**

Elastic critical torsional-flexural buckling force $N_{cr,TF} =$ **4180** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **4180** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})} =$ **0.501**

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

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_T =$ **0.34**

Buckling reduction determination factor $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) =$ **0.677**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1) =$ **0.884**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **928.5** kN

$N_{Ed} / N_{b,T,Rd} =$ **0.063**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f =$ **187.6** mm $\eta =$ **1.000**

$h_w / t_w = 29.3 = 31.7 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{y,Ed}$ = **38.7** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1458** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **231.4** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.167**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **65.9** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **86.5** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.762**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.823**
	$C_1 = 1 / k_c^2$ = **1.478**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m3\_s1\_seg1\_B}$ = **1200** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **866.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.316**
Limiting slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.4**
	$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check bending and axial force - Section 6.2.9

Bending and axial force check - eq.6.33 & eq.6.34	$N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **165.1** kN
	$N_{Ed} / N_{y,lim}$ = **0.356**

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	ψ_y = 13.09 kNm / -65.866 kNm = **-0.199**
	α_y = 13.982 kNm / -65.866 kNm = **-0.212**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = -25.433 kNm / -65.866 kNm = **0.386**
	α_{LT} = -44.142 kNm / -65.866 kNm = **0.670**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.736**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **86.5** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **24.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **1050.8** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.418**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.991**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.39**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.826**
	PASS - Combined bending and compression checks are satisfied

Check design 4623 mm along span

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **23.6** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **86.5** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.273**

PASS - Design bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3	ψ_y = 13.09 kNm / -65.866 kNm = **-0.199**
	α_y = 13.982 kNm / -65.866 kNm = **-0.212**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 15.693 kNm / -25.433 kNm = **-0.617**
	α_{LT} = 19.251 kNm / -25.433 kNm = **-0.757**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **86.5** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **24.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **1050.8** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.415**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.977**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.172**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.325**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design 5668 mm along span

Check y-y axis deflection - Section 7.2.1

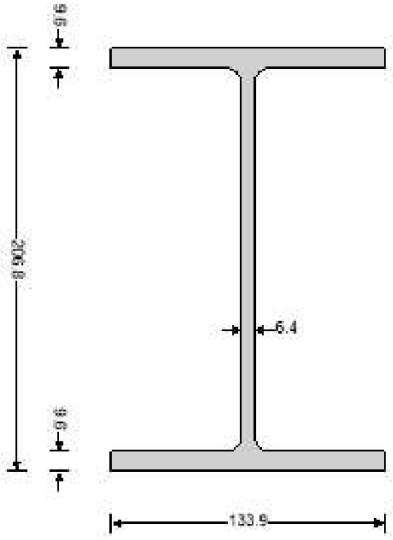
Maximum deflection	δ_y = **28.7** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m3\_s1} / 100$ = **62.1** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.462**

PASS - Allowable deflection exceeds design deflection

4 design

Section details

Section type	UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **9.6** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 205.8 mm
Section breadth, b, 133.9 mm
Flange thickness, t_f, 9.6 mm
Web thickness, t_w, 6.4 mm
Root radius, r, 7.6 mm
Area of section, A, 3821 mm²
Radius of gyration about y-axis, i_y, 87.051 mm
Radius of gyration about z-axis, i_z, 31.728 mm
Elastic section modulus about y-axis, $W_{el,y}$, 280036 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 57454 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 314365 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 88224 mm³
Second moment of area about y-axis, I_y, 28955684 mm⁴
Second moment of area about z-axis, I_z, 3846542 mm⁴

Lateral restraint

Upper flange has full lateral restraint

Lower flange has lateral restraint at supports plus 1200 & 6000 mm

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength)

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} =$ **0.92**

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

Width of section

$c = d =$ **172.4** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.598**

$c / t_w = 26.9 = 29.1 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$c = (b - t_w - 2 \times r) / 2 =$ **56.1** mm

$c / t_f = 5.8 = 6.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force: $N_{Ed} =$ **47** kN

Design resistance of section - eq 6.10: $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **1050.8** kN

$N_{Ed} / N_{c,Rd} =$ **0.045**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length: $L_{cr,y} = L_{m4\_s1} =$ **6210** mm

Critical buckling force: $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **1556.4** kN

Slenderness ratio for buckling - eq 6.50: $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.822**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2: a

Imperfection factor - Table 6.1: $\alpha_y =$ **0.21**

Buckling reduction determination factor: $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.903**

Buckling reduction factor - eq 6.49: $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.783**

Design buckling resistance - eq 6.47: $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **822.9** kN

N_{Ed} / $N_{b,y,Rd}$ = **0.057**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m4\_s1\_seg1\_R}$ = **1200** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **92.7** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **7239.5** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **4180** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.501**

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

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	α_T = **0.34**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.677**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.884**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **928.5** kN
	N_{Ed} / $N_{b,T,Rd}$ = **0.051**

PASS - Design buckling resistance exceeds design compression

Check design 1396 mm along span

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **19.4** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **86.5** kNm
	$M_{y,Ed}$ / $M_{c,y,Rd}$ = **0.225**

PASS - Design bending 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	$N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **165.1** kN
	N_{Ed} / $N_{y,lim}$ = **0.301**

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	ψ_y = 12.131 kNm / -67.339 kNm = **-0.180**
	α_y = 8.49 kNm / -67.339 kNm = **-0.126**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 19.282 kNm / -59.945 kNm = **-0.322**
	α_{LT} = 1.235 kNm / -59.945 kNm = **-0.021**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **86.5** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **24.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **1050.8** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.415**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.977**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.154**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.279**

PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web: $h_w = h - 2 \times t_f =$ **187.6** mm $\eta =$ **1.000**

$h_w / t_w = 29.3 = 31.7 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force: $V_{y,Ed} =$ **36** kN

Shear area - cl 6.2.6(3): $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1458** mm²

Design shear resistance - cl 6.2.6(2): $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **231.4** kN

$V_{y,Ed} / V_{c,y,Rd} =$ **0.156**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment: $M_{y,Ed} =$ **67.3** kNm

Design bending resistance moment - eq 6.13: $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **86.5** kNm

$M_{y,Ed} / M_{c,y,Rd} =$ **0.779**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6: $k_c =$ **0.97**

$C_1 = 1 / k_c^2 =$ **1.064**

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

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

Unrestrained effective length: $L = 1.0 \times L_{m4\_s1\_seg3\_B} =$ **210** mm

Elastic critical buckling moment: $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **19063.4** kNm

Slenderness ratio for lateral torsional buckling: $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} =$ **0.067**

Limiting slenderness ratio: $\bar{\lambda}_{LT,0} =$ **0.4**

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3: $\psi_y =$ 12.131 kNm / -67.339 kNm = **-0.180**

$\alpha_y =$ 8.49 kNm / -67.339 kNm = **-0.126**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4) =$ **0.400**

$\psi_{LT} =$ -59.945 kNm / -67.339 kNm = **0.890**

$\alpha_{LT} =$ -63.601 kNm / -67.339 kNm = **0.944**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4) =$ **0.956**

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

Characteristic moment resistance: $M_{y,Rk} = W_{pl,y} \times f_y =$ **86.5** kNm

Characteristic moment resistance: $M_{z,Rk} = W_{pl,z} \times f_y =$ **24.3** kNm

Characteristic resistance to normal force: $N_{Rk} = A \times f_y =$ **1050.8** kN

Interaction factors: $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.418**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.994**

Interaction formulae - eq 6.61 & eq 6.62: $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.398**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.846**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection — δ_y = **24.1** mm

Allowable deflection — $\delta_{y,Allowable} = L_{m4\_s1} / 100$ = **62.1** mm

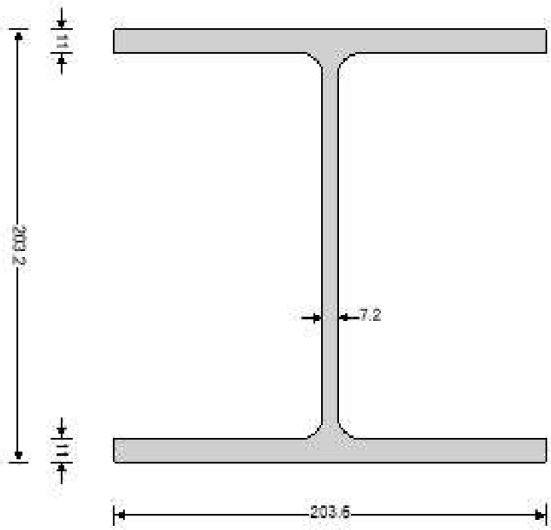
$\delta_y / \delta_{y,Allowable}$ = **0.388**

PASS - Allowable deflection exceeds design deflection

Member5 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **11** mm
Nominal yield strength	f_y = **355** N/mm²
Nominal ultimate tensile strength	f_u = **470** N/mm²
Modulus of elasticity	E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, i_y, 88.19 mm
Radius of gyration about z-axis, i_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230884 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Right flange has full lateral restraint

Left flange has lateral restraint at supports only

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section — c = d = **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.563**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 =$ **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force — $N_{Ed} =$ **50.1** kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **2085** kN

$N_{Ed} / N_{c,Rd} =$ **0.024**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length — $L_{cr,y} = L_{m5\_s1} =$ **2300** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **17896.7** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.341**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — b

Imperfection factor - Table 6.1 — $\alpha_y =$ **0.34**

Buckling reduction determination factor — $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.582**

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.949**

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **1978.1** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.025**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length — $L_{cr,T} = L_{m5\_s1\_seg1\_R} =$ **2300** mm

Distance from shear centre to centroid in y axis — $y_0 =$ **0.0** mm

Distance from shear centre to centroid in z axis — $z_0 =$ **0.0** mm

Radius of gyration — $i_0 = \sqrt{(i_y^2 + i_z^2)} =$ **102.0** mm

Elastic critical torsional buckling force — $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2 =$ **7097.8** kN

Torsion factor — $\beta_T = 1 - (y_0 / i_0)^2 =$ **1**

Elastic critical torsional-flexural buckling force — $N_{cr,TF} =$ **4180** kN

Elastic critical buckling force — $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **4180** kN

Slenderness ratio for torsional buckling - eq 6.52 — $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})} =$ **0.706**

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 — $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) =$ **0.873**

Buckling reduction factor - eq 6.49 — $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1) =$ **0.721**

Design buckling resistance - eq 6.47 — $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **1502.9** kN

$N_{Ed} / N_{b,T,Rd} =$ **0.033**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f =$ **181.2** mm $\eta =$ **1.000**

$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{y,Ed}$ = **48.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.138**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **67.3** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.381**
	PASS - Design bending 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
	$N_{Ed} / N_{y,lim}$ = **0.216**
	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	ψ_y = 43.2 kNm / -67.339 kNm = **-0.642**
	α_y = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4)$ = **0.400**
	ψ_{LT} = 43.2 kNm / -67.339 kNm = **-0.642**
	α_{LT} = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** 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**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.988**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.178**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.407**
	PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm η = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **48.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.138**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **43.2** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm

$M_{y,Ed}$ / $M_{c,y,Rd}$ = **0.245**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.65**
	$C_1 = 1 / k_c^2$ = **2.367**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m5\_s1\_seg1\_T}$ = **2300** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **1584.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.334**
Limiting slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3	ψ_y = 43.2 kNm / -67.339 kNm = **-0.642**
	α_y = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4)$ = **0.400**
	ψ_{LT} = 43.2 kNm / -67.339 kNm = **-0.642**
	α_{LT} = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl.y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl.z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** 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**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.988**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.124**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.273**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection	δ_y = **15.2** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m5\_s1} / 100$ = **23** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.662**

PASS - Allowable deflection exceeds design deflection

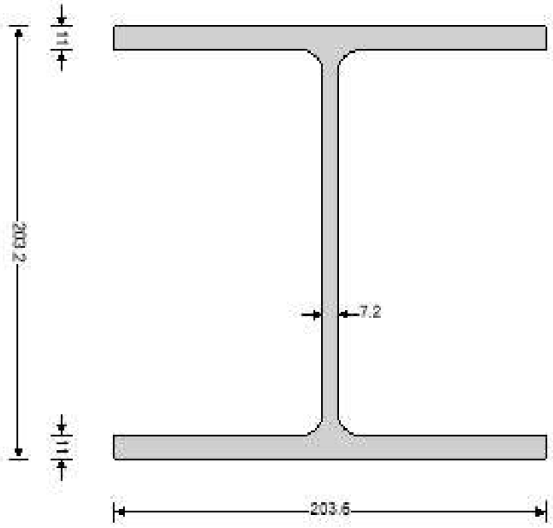
Member6 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **11** mm
Nominal yield strength	f_y = **355** N/mm²

Nominal ultimate tensile strength f_u = **470** N/mm²

Modulus of elasticity E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, I_y, 88.19 mm
Radius of gyration about z-axis, I_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230864 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section c = d = **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.569**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force N_{Ed} = **54.6** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = A \times f_y / \gamma_{M0}$ = **2085** kN

$N_{Ed} / N_{c,Rd}$ = **0.026**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length $L_{cr,y} = L_{m6\_s1}$ = **3400** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **8189.7** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.505**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 α_y = **0.34**

Buckling reduction determination factor $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.679**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.882**

Design buckling resistance - eq 6.47	$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **1839.3** kN
	$N_{Ed} / N_{b,y,Rd}$ = **0.03**
	PASS - Design buckling resistance exceeds design compression

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

Critical buckling length	$L_{cr,z} = L_{m6\_s1\_seg1}$ = **3400** mm
Critical buckling force	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **2775.8** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **0.867**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_z = **0.49**
Buckling reduction determination factor	$\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **1.039**
Buckling reduction factor - eq 6.49	$\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.62**
Design buckling resistance - eq 6.47	$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **1293.6** kN
	$N_{Ed} / N_{b,z,Rd}$ = **0.042**
	PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m6\_s1\_seg1\_R}$ = **3400** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **102.0** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **4180** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **4180** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.706**

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

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_T = **0.49**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.873**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.721**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **1502.9** kN
	$N_{Ed} / N_{b,T,Rd}$ = **0.036**
	PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm $\quad \eta$ = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **13.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.038**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed}$ = **31.2** kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.177**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.676**

$C_1 = 1 / k_c^2$ = **2.19**

Poissons ratio — ν = **0.3**

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

Unrestrained effective length — $L = 1.0 \times L_{m6\_s1\_seg1\_T}$ = **3400** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **761.3** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.482**

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 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.601**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.968**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.871**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**

Design buckling resistance moment - eq 6.55 — $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.177**

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

$N_{Ed} / N_{y,lim}$ = **0.236**

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 — ψ_y = -13.355 kNm / 31.171 kNm = **-0.428**

α_y = 8.908 kNm / 31.171 kNm = **0.286**

$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4)$ = **0.429**

ψ_{LT} = -13.355 kNm / 31.171 kNm = **-0.428**

α_{LT} = 8.908 kNm / 31.171 kNm = **0.286**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4)$ = **0.429**

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

Characteristic moment resistance — $M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm

Characteristic moment resistance — $M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm

Characteristic resistance to normal force — $N_{Rk} = A \times f_y$ = **2085** kN

Interaction factors — $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.432**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.980**

Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.109**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.221**
	PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm η = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **13.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.038**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **13.4** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.076**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.676**
	$C_1 = 1 / k_c^2$ = **2.19**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m6\_s1\_seg1\_B}$ = **3400** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **761.3** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.482**
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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.601**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.968**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.871**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.076**
	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	ψ_y = -13.355 kNm / 31.171 kNm = **-0.428**
	α_y = 8.908 kNm / 31.171 kNm = **0.286**

C_{my} = max(0.6 + 0.4 × ψ_y, 0.4) = **0.429**

ψ_{LT} = -13.355 kNm / 31.171 kNm = **-0.428**

α_{LT} = 8.908 kNm / 31.171 kNm = **0.286**

C_{mLT} = max(0.6 + 0.4 × ψ_{LT}, 0.4) = **0.429**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.433**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.979**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.065**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.12**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection	δ_y = **5.1** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m6\_s1} / 100$ = **34** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.149**

PASS - Allowable deflection exceeds design deflection

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Our ref: 07229E_sH_SE_RP_003

Dr Sally Murrall-Smith PhD
Totnes Renewable Energy Society
107 High Street
Totnes
TQ9 5SN

21st January 2026

Dear Sally,

Structural Check at Dart Marine Park, Steamer Quay Road, Totnes, TQ9 5AL – Building C

As requested, and commissioned, we have carried out a structural survey of the portal frame building at Dart Marine Park, carried out calculations and prepared this report, for the purpose of justifying the proposed PV panel installation to the roof.

The building is of a two storey portalised steel frame, we could not measure the floor steel due to the finishes of the building, we requested 'As-Built' drawings, but these haven't been available. However, we are confident that the compressive utilisation of the steel columns is not at question, more the overall stability, as such we have included a tie beam at floor level but not analysed this element.

The portal steel frame has 203 x 133 x 30 UB Rafters and 203 UC 46 Columns, the portal frames are at 5.7 metre centres. The dimension of the portal frame is shown in the calculations within this report.

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VAT Registration Nº GB 190 6091 09

The proposed PV system is fixed to the roof, loadings are taken as 0.15kN/m^2 which is applied on top of the snow loading at 0.6kN/m^2 and the dead weight loading of 0.5kN/m^2 including services.

Structural calculations have been carried out to check the capacity of these members, and determine adequacy of the existing structure for the proposed PV panel instillation.

We are pleased to confirm that the steel frame member sizes are adequate to support the additional vertical loading of the proposed installation.

Therefore, we can confirm that installing the new roof panels upon the existing roof of this structure will not have a structurally detrimental effect.

This letter confirms cover under our PI should any failure of the steel frame occur due to inadequate capacity.

This letter is not intended to and does not relate to the adequacy or assessment of the fixing system to the Photovoltaics to the roof.

Yours sincerely,



Mark Harris MIStructE
Director
structureHaus Ltd

Appendix A – Information Provided and Photos

Appendix B – Structural Calculations

APPENDIX A – INFORMATION PROVIDED & SITE PHOTOS



Proposed PV layout



Site Visit Photo

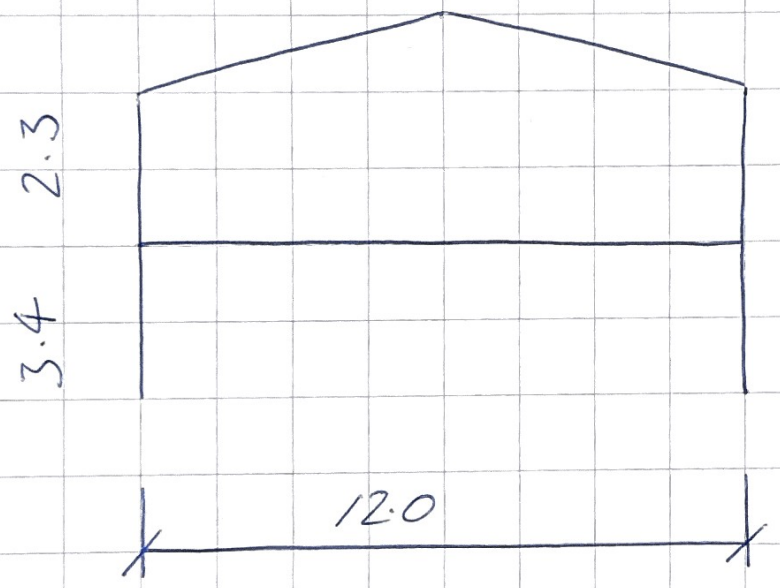


Site Visit Photo

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APPENDIX B – STRUCTURAL CALCULATIONS

Building C



@ 5.7 m c/c's

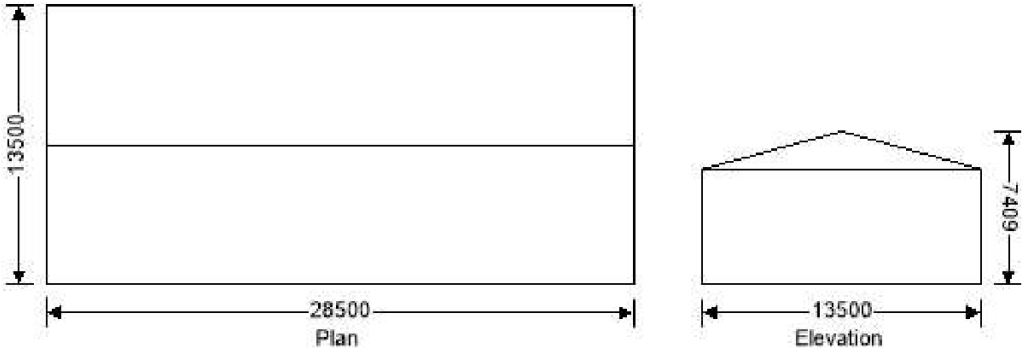
Rafters 203 × 133 × 30 UB

Columns 203 UC 46

WIND LOADING (BS6399)

In accordance with BS6399

Tedds calculation version 3.0.19



Building data

Type of roof	Duopitch
Length of building	L = **28500** mm
Width of building	W = **13500** mm
Height to eaves	H = **5600** mm
Pitch of roof	α_0 = **15.0** deg
Reference height	H_r = **7409** mm

Dynamic classification

Building type factor (Table 1)	K_b = **1.0**
Dynamic augmentation factor (1.6.1)	$C_r = [K_b \times (H_r / (0.1\ m))^{0.75}] / (800 \times \log(H_r / (0.1\ m))) =$ **0.02**

Site wind speed

Location	Totnes
Basic wind speed (Figure 6 BS6399:Pt 2)	V_b = **22.5** m/s
Site altitude	Δs = **1** m
Upwind distance from sea to site	d_{sea} = **10** km
Direction factor	S_d =**0.82**
Seasonal factor	S_s = **1.00**
Probability factor	S_p = **1.00**
Critical gap between buildings	g = **5000** mm
Topography not significant	
Altitude factor	$S_a = 1 + 0.001 \times \Delta s$ / 1m = **1.00**
Site wind speed	$V_s = V_b \times S_a \times S_d \times S_s \times S_p$ = **18.5** m/s
Terrain category	Sea
Displacement height (sheltering effect excluded)	H_d = 0mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

Dynamic pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought)	H_{ref} = **5600**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **5600**mm
Fetch factor (Table 22)	S_c = **0.959**

Turbulence factor (Table 22)	S_t = **0.190**
Gust peak factor	g_t = **3.44**
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.59**
Effective wind speed	$V_e = V_s \times S_b$ = **29.3** m/s
Dynamic pressure	q_s = 0.613 kg/m^3 × V_e^2 = **0.527** kN/m^2

Dynamic pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought)	H_{ref} = **7409**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **7409**mm
Fetch factor (Table 22)	S_c = **1.005**
Turbulence factor (Table 22)	S_t = **0.185**
Gust peak factor	g_t = **3.44**
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.64**
Effective wind speed	$V_e = V_s \times S_b$ = **30.4** m/s
Dynamic pressure	q_s = 0.613 kg/m^3 × V_e^2 = **0.566** kN/m^2

Size effect factors

Diagonal dimension for gablewall	a_{eg} = **14.6** m
External size effect factor gablewall	C_{aeg} = **0.903**
Diagonal dimension for side wall	a_{es} = **29.0** m
External size effect factor side wall	C_{aes} = **0.841**
Diagonal dimension for roof	a_{er} = **29.3** m
External size effect factor roof	C_{aer} = **0.840**
Room/storey volume for internal size effect factor	V_i = **1740.000** m^3
Diagonal dimension for internal size effect factors	$a_i = 10 \times (V_i)^{1/3}$ = **120.277** m
Internal size effect factor	C_{ai} = **0.712**

Pressures and forces

Net pressure	$p = q_s \times c_{pe} \times C_{ae} - q_s \times c_{pi} \times C_{ai}$
Net force	$F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.10	0.57	0.840	-0.60	22.73	-13.71
B (-ve)	-0.80	0.57	0.840	-0.46	20.99	-9.67
C (-ve)	-0.40	0.57	0.840	-0.27	155.44	-42.06
E (-ve)	-1.30	0.57	0.840	-0.70	22.73	-15.87
F (-ve)	-0.90	0.57	0.840	-0.51	20.99	-10.66
G (-ve)	-0.50	0.57	0.840	-0.32	155.44	-49.44

Total vertical net force	$F_{w,v}$ = **-136.59** kN
Total horizontal net force	$F_{w,h}$ = **2.73** kN

Walls load case 1 - Wind 0, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.40	0.57	0.903	-0.80	17.77	-14.19
B	-0.83	0.57	0.903	-0.51	70.04	-35.51
w	0.73	0.53	0.841	0.25	159.60	39.78
l	-0.50	0.53	0.841	-0.30	159.60	-47.29

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-47.3** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **39.8** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **77.6** kN

Roof load case 2 - Wind 0, c_{pi} -0.3, +c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A (+ve)	0.20	0.57	0.840	0.22	22.73	4.91
B (+ve)	0.20	0.57	0.840	0.22	20.99	4.53
C (+ve)	0.20	0.57	0.840	0.22	155.44	33.55
E (+ve)	-1.30	0.57	0.840	-0.50	22.73	-11.29
F (+ve)	-0.90	0.57	0.840	-0.31	20.99	-6.44
G (+ve)	-0.50	0.57	0.840	-0.12	155.44	-18.14

Total vertical net force $F_{w,v}$ = **6.87** kN

Total horizontal net force $F_{w,h}$ = **20.41** kN

Walls load case 2 - Wind 0, c_{pi} -0.3, +c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.40	0.57	0.903	-0.60	17.77	-10.61
B	-0.83	0.57	0.903	-0.31	70.04	-21.41
w	0.73	0.53	0.841	0.44	159.60	69.70
l	-0.50	0.53	0.841	-0.11	159.60	-17.38

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-17.4** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **69.7** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **92.9** kN

Roof load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.60	0.57	0.840	-0.84	9.43	-7.93
B (-ve)	-1.50	0.57	0.840	-0.79	9.43	-7.48
C (-ve)	-0.60	0.57	0.840	-0.37	75.47	-27.59
D (-ve)	-0.40	0.57	0.840	-0.27	303.98	-82.25

Total vertical net force $F_{w,v}$ = **-120.99** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.54	0.53	0.841	-0.75	12.54	-9.47
B	-0.88	0.53	0.841	-0.46	50.18	-23.28
C	-0.81	0.53	0.841	-0.44	96.88	-42.19
w	0.61	0.57	0.903	0.23	87.81	20.41
l	-0.50	0.57	0.903	-0.34	87.81	-29.50

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-29.5** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **20.4** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **43.1** kN

Roof load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.60	0.57	0.840	-0.64	9.43	-6.03
B (-ve)	-1.50	0.57	0.840	-0.59	9.43	-5.58
C (-ve)	-0.60	0.57	0.840	-0.16	75.47	-12.39
D (-ve)	-0.40	0.57	0.840	-0.07	303.98	-21.04

Total vertical net force $F_{w,v}$ = **-43.51** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

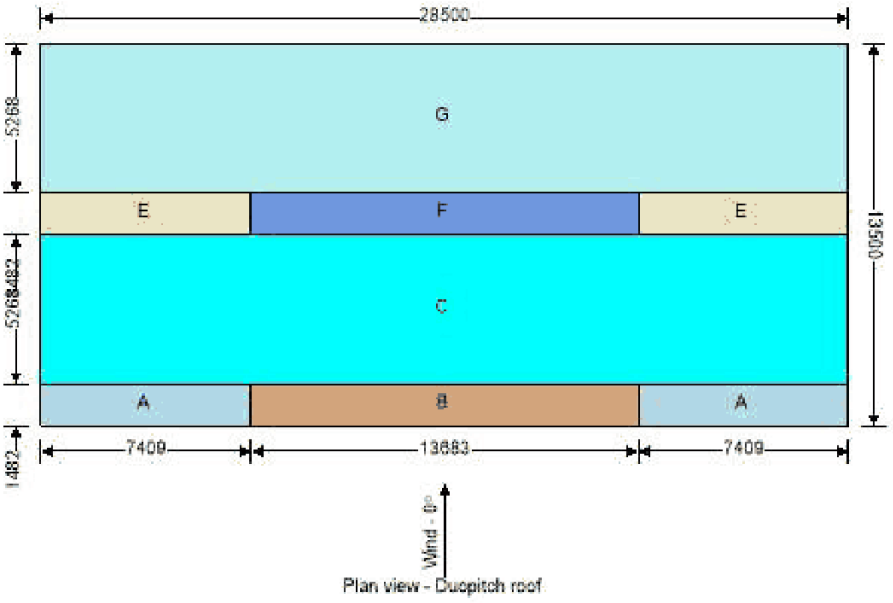
Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.54	0.53	0.841	-0.57	12.54	-7.12
B	-0.88	0.53	0.841	-0.28	50.18	-13.87
C	-0.81	0.53	0.841	-0.25	96.88	-24.03
w	0.61	0.57	0.903	0.43	87.81	38.09
l	-0.50	0.57	0.903	-0.13	87.81	-11.82

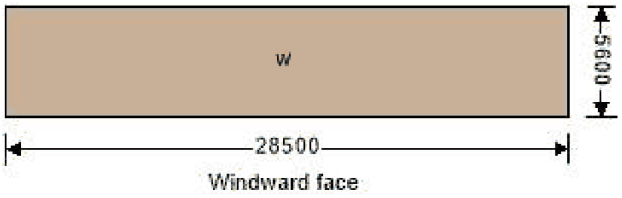
Overall loading

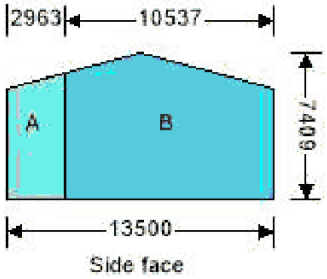
Equiv leeward net force for overall section — $F_l = F_{w,wl}$ = **-11.8** kN

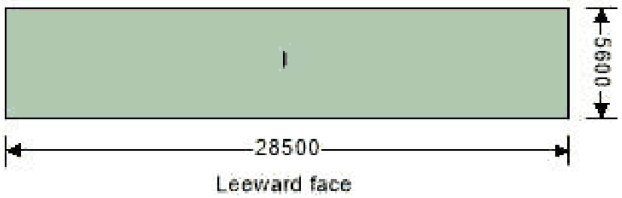
Net windward force for overall section — $F_w = F_{w,ww}$ = **38.1** kN

Overall loading overall section — $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **43.1** kN

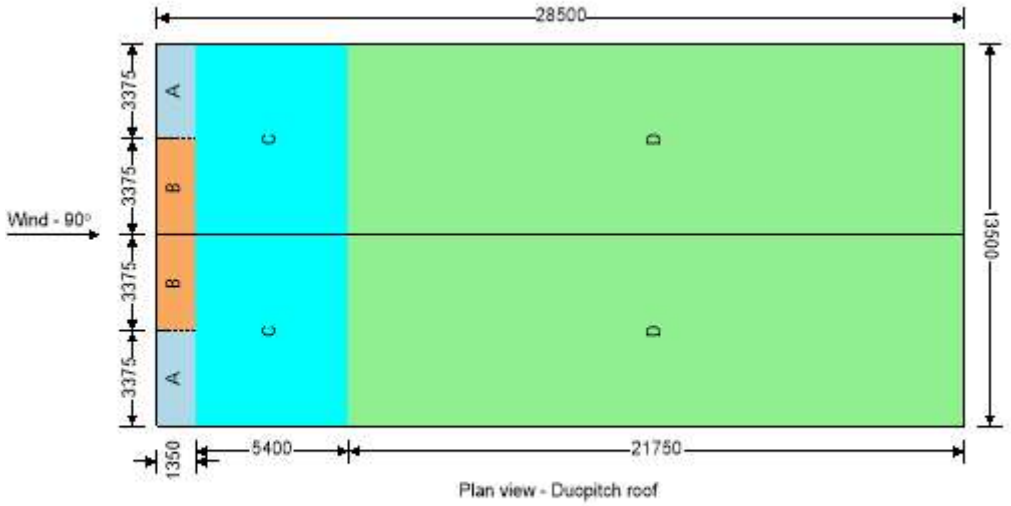




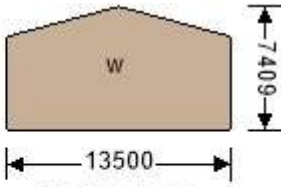




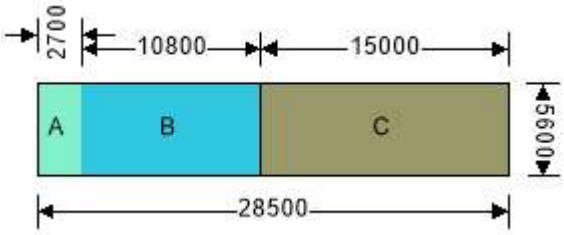
structureHaus Ltd	Project: Dart Marine Park, Totnes				Job no.: 07229E	
	Calcs for: Wind				Start page no./Revision: 7	
	Calcs by: MDH	Calcs date: 11/11/2025	Checked by: DALP	Checked date:	Approved by:	Approved date:



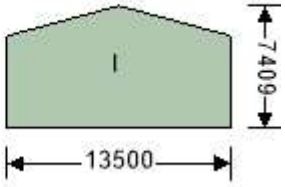
Plan view - Duopitch roof



Windward face

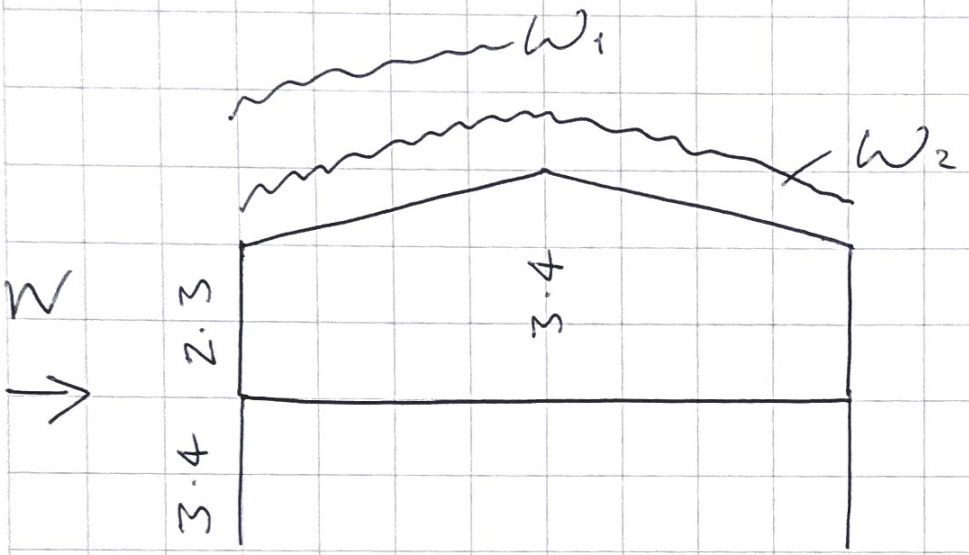


Side face



Leeward face

Building C



@ 5.7m c/c's

$W = 1.12 \times 5.7 = 6.38\ kN/m$

$W_1 = 0.12 \times 5.7 = 0.68\ kN/m$

$W_2 = 0.5 \times 5.7 = 2.85\ kN/m$

$0.6 = 3.42\ kN/m$

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

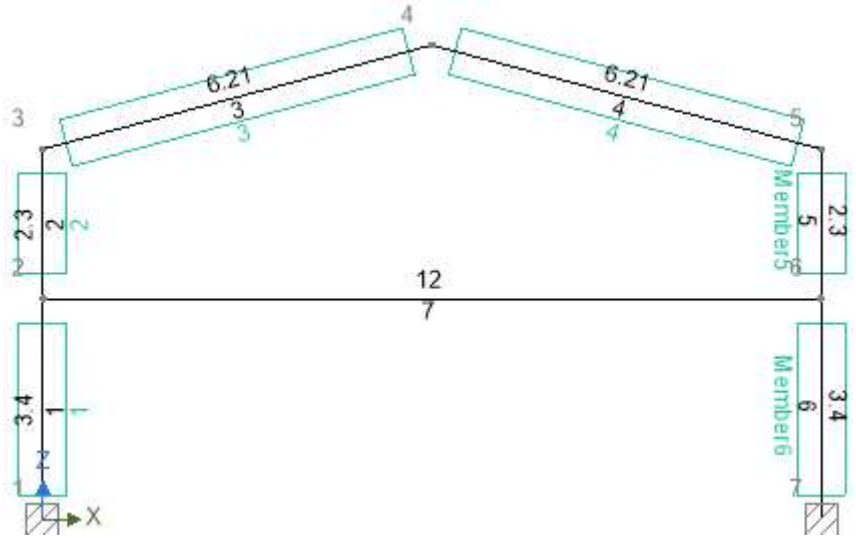
Tedds calculation version 4.5.06

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (m) - Steel (EC3)



Materials

Name	Density (kg/m³)	Youngs Modulus kN/mm²	Shear Modulus kN/mm²	Thermal Coefficient °C⁻¹
Steel (EC3)	7850	210	80.8	0.000012

Sections

Name	Area (cm²)	Moment of inertia Major (cm⁴)	Moment of inertia Minor (cm⁴)	Shear area parallel to Minor (cm²)	Shear area parallel to Major (cm²)
UKC 203x203x46	58.7	4567.8	1548.2	14.6	40.3
UB 203x133x30	38.2	2895.6	384.7	13.2	23.1

Nodes

Node	Co-ordinates X (m)	Co-ordinates Z (m)	Freedom X	Freedom Z	Freedom Rot.	Coordinate system Name	Coordinate system Angle (°)	Spring X (kN/m)	Spring Z (kN/m)	Spring Rot. kNm/°
1	0	0	Fixed	Fixed	Fixed		0	0	0	0
2	0	3.4	Free	Free	Free		0	0	0	0
3	0	5.7	Free	Free	Free		0	0	0	0
4	6	7.3	Free	Free	Free		0	0	0	0
5	12	5.7	Free	Free	Free		0	0	0	0
6	12	3.4	Free	Free	Free		0	0	0	0
7	12	0	Fixed	Fixed	Fixed		0	0	0	0

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	3.4	1	2	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
2	2.3	2	3	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
3	6.21	3	4	UB 203x133x30	Steel (EC3)	Fixed	Fixed	Fixed	
4	6.21	4	5	UB 203x133x30	Steel (EC3)	Fixed	Fixed	Fixed	
5	2.3	5	6	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
6	3.4	6	7	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	
7	12	2	6	UKC 203x203x46	Steel (EC3)	Fixed	Fixed	Fixed	

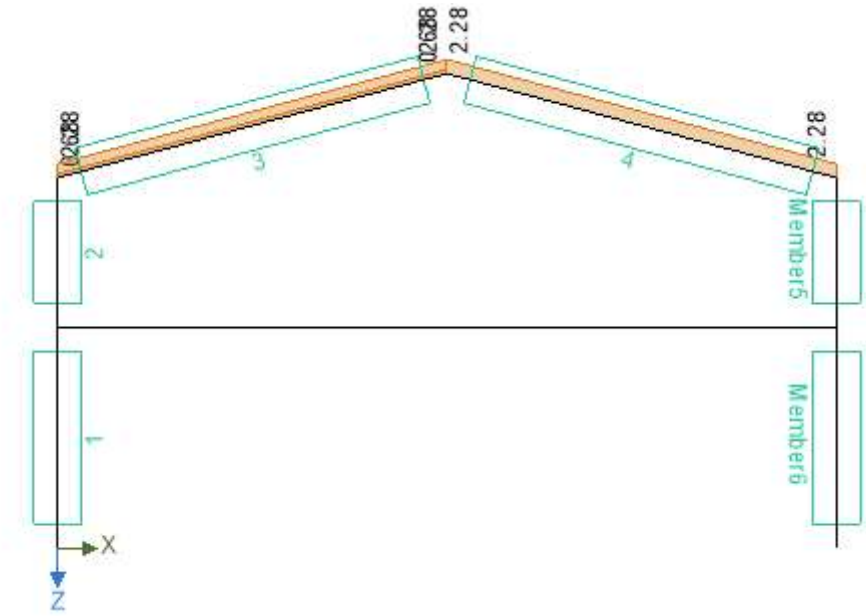
Members

Name	Elements	
	Start	End
1	1	1
2	2	2
3	3	3
4	4	4
Member5	5	5
Member6	6	6

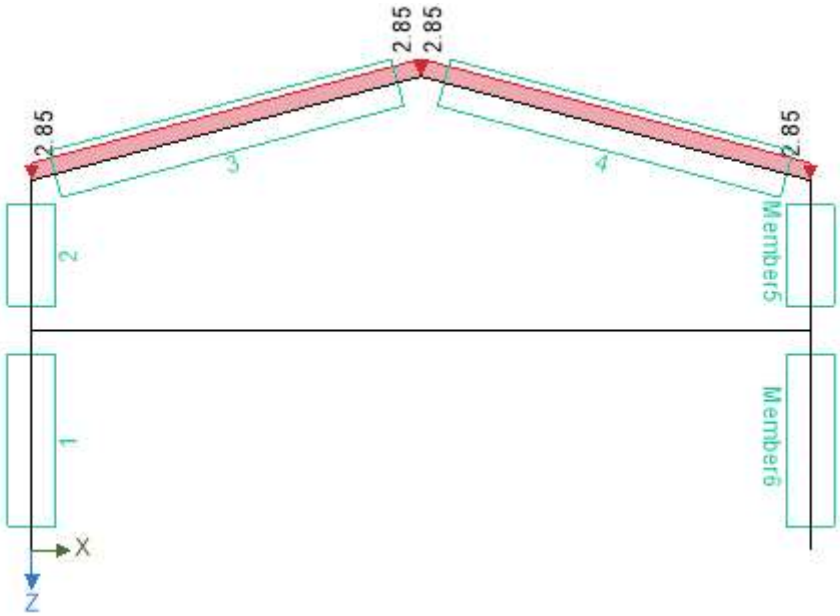
Loading

Self weight included

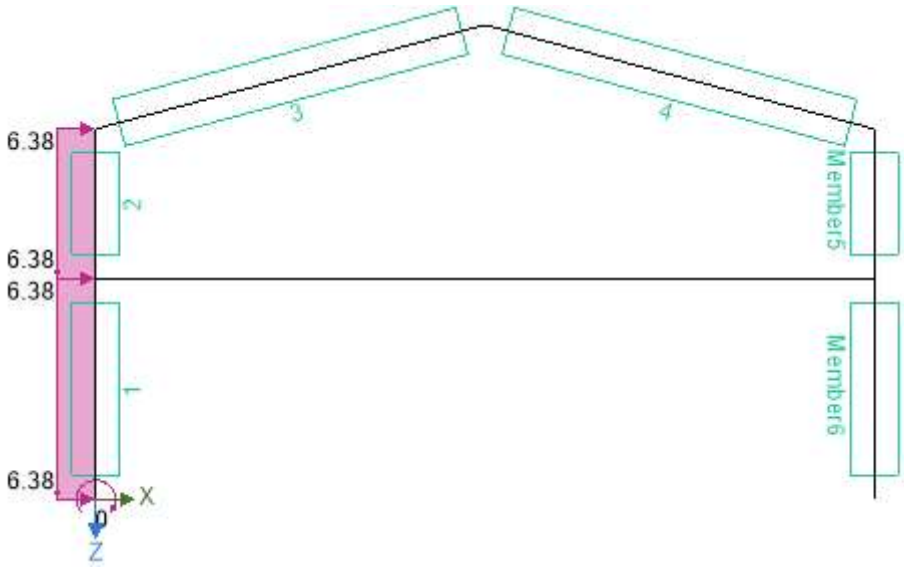
Permanent - Loading (kN/m)



Imposed - Loading (kN/m)



Wind - Loading (kN/m,kNm)



Load combination factors

Load combination	Self Weight	Permanent	Imposed	Wind
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50	
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00	
1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	1.50	0.75
1.0G + 1.0Q + 0.5S + 0.5W (Service)	1.00	1.00	1.00	0.50

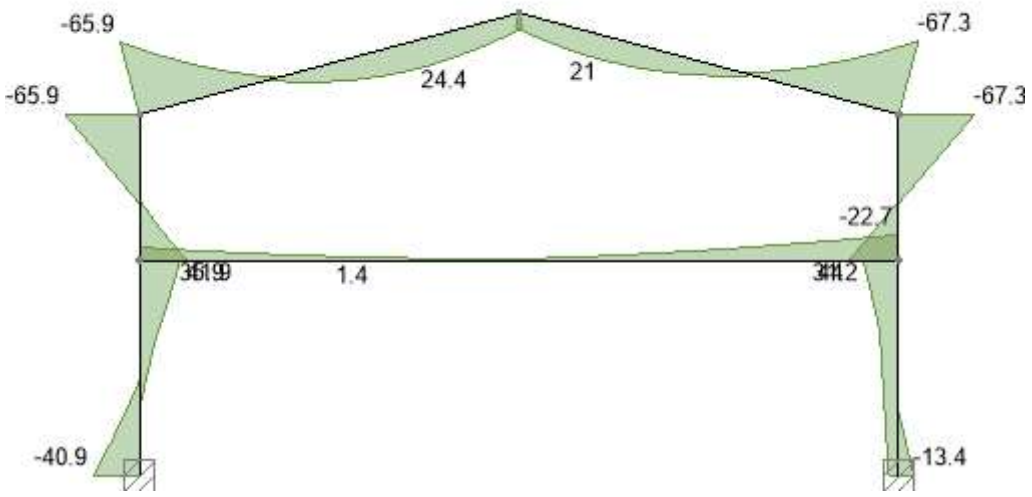
Member Loads

Member	Load case	Load Type	Orientation	Description
3	Permanent	UDL	GlobalZ	0.68 kN/m
3	Permanent	UDL	GlobalZ	2.28 kN/m
4	Permanent	UDL	GlobalZ	2.28 kN/m
3	Imposed	UDL	GlobalZ	2.85 kN/m
4	Imposed	UDL	GlobalZ	2.85 kN/m
1	Wind	Point Moment	GlobalY	0 kNm at 0 m
1	Wind	UDL	GlobalX	6.38 kN/m
2	Wind	UDL	GlobalX	6.38 kN/m

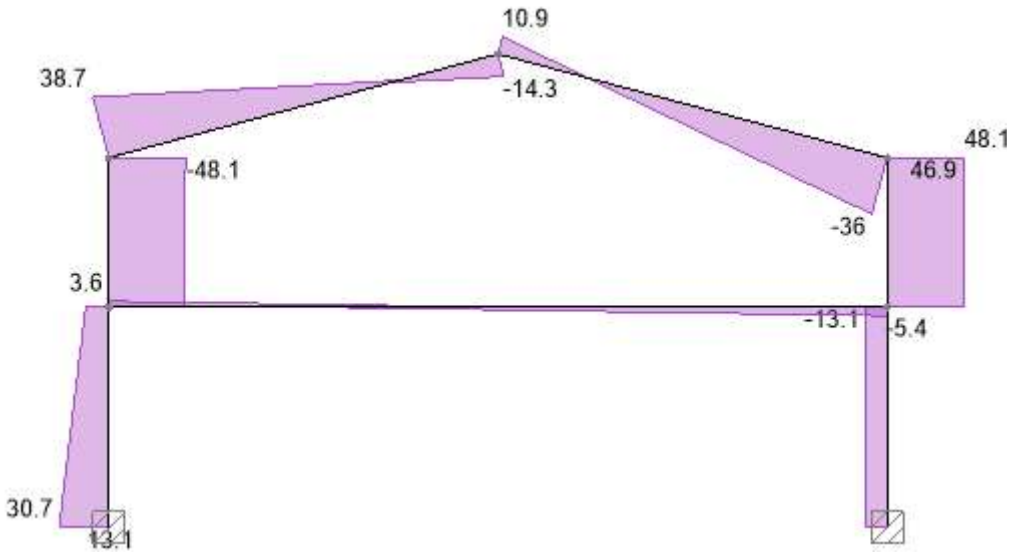
Results

Forces

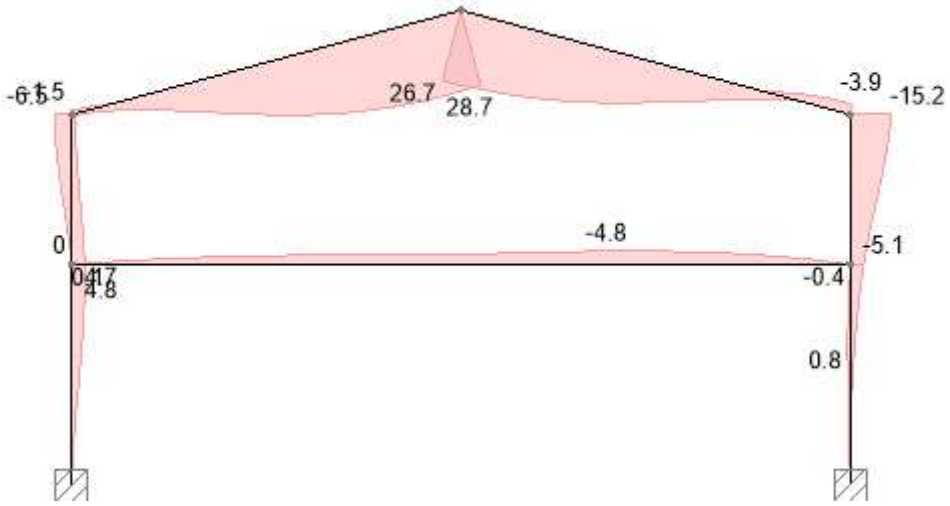
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



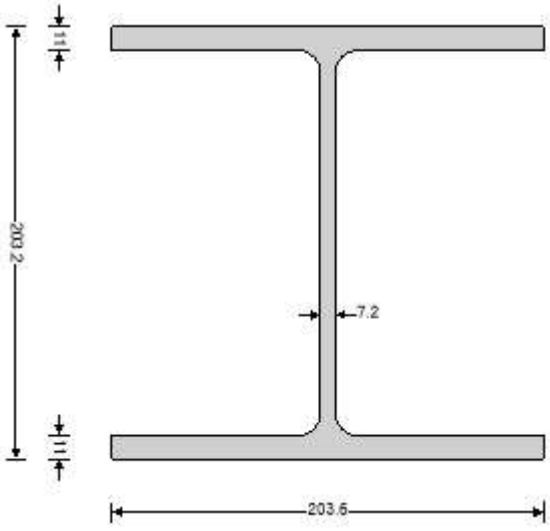
Partial factors - Section 6.1

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**

1 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	t_{nom} = max(t_f, t_w) = **11** mm
Nominal yield strength	f_y = **355** N/mm²
Nominal ultimate tensile strength	f_u = **470** N/mm²
Modulus of elasticity	E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, i_y, 88.19 mm
Radius of gyration about z-axis, i_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230864 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section

$c = d =$ **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.570**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$c = (b - t_w - 2 \times r) / 2 =$ **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force — $N_{Ed} =$ **57.2** kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **2085** kN

$N_{Ed} / N_{c,Rd} =$ **0.027**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length — $L_{cr,y} = L_{m1\_s1} =$ **3400** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **8189.7** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.505**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — b

Imperfection factor - Table 6.1 — $\alpha_y =$ **0.34**

Buckling reduction determination factor — $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.679**

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.882**

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **1839.3** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.031**

PASS - Design buckling resistance exceeds design compression

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

Critical buckling length — $L_{cr,z} = L_{m1\_s1\_seg1} =$ **3400** mm

Critical buckling force — $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **2775.8** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **0.867**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — c

Imperfection factor - Table 6.1 — $\alpha_z =$ **0.49**

Buckling reduction determination factor — $\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **1.039**

Buckling reduction factor - eq 6.49 — $\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1) =$ **0.62**

Design buckling resistance - eq 6.47 — $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **1293.6** kN

$N_{Ed} / N_{b,z,Rd} =$ **0.044**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m1\_s1\_seg1\_R}$ = **3400** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **102.0** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **4180** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **4180** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.706**

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

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_T = **0.49**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.873**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.721**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **1502.9** kN
	$N_{Ed} / N_{b,T,Rd}$ = **0.038**
	PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm η = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **30.7** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.088**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **40.9** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.231**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.634**
	$C_1 = 1 / k_c^2$ = **2.49**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m1\_s1\_seg1\_B}$ = **3400** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **865.6** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.452**
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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.585**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.980**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.861**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.231**

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
	$N_{Ed} / N_{y,lim}$ = **0.247**

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	ψ_y = 35.916 kNm / -40.856 kNm = **-0.879**
	α_y = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 35.916 kNm / -40.856 kNm = **-0.879**
	α_{LT} = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.404**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.974**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.126**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.274**

PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm η = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{y,Ed}$ = **14.4** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm^2
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.042**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **35.9** kNm

Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.203**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.634**
	$C_1 = 1 / k_c^2$ = **2.49**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2
Unrestrained effective length	$L = 1.0 \times L_{m1\_s1\_seg1\_T}$ = **3400** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **865.6** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.452**
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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.585**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.980**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.861**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.203**
	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	ψ_y = 35.916 kNm / -40.856 kNm = **-0.879**
	α_y = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 35.916 kNm / -40.856 kNm = **-0.879**
	α_{LT} = 4.444 kNm / -40.856 kNm = **-0.109**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.404**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.975**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.114**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.245**
	PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design 3199 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection — δ_y = **4.8** mm

Allowable deflection — $\delta_{y,Allowable} = L_{m1\_s1} / 100$ = **34** mm

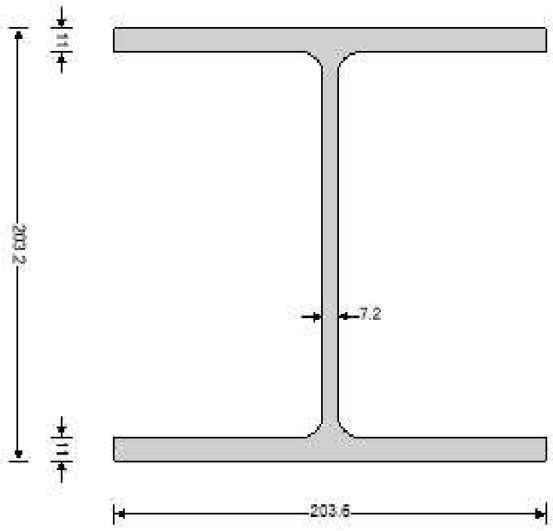
$\delta_y / \delta_{y,Allowable}$ = **0.14**

PASS - Allowable deflection exceeds design deflection

2 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **11** mm
Nominal yield strength	f_y = **355** N/mm²
Nominal ultimate tensile strength	f_u = **470** N/mm²
Modulus of elasticity	E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, i_y, 88.19 mm
Radius of gyration about z-axis, i_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230884 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section — c = d = **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.566**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force	N_{Ed} = **54** kN
Design resistance of section - eq 6.10	$N_{c,Rd} = A \times f_y / \gamma_{M0}$ = **2085** kN
	$N_{Ed} / N_{c,Rd}$ = **0.026**
	PASS - Design compression resistance exceeds design compression

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

Critical buckling length	$L_{cr,y} = L_{m2\_s1}$ = **2300** mm
Critical buckling force	$N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **17896.7** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.341**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	α_y = **0.34**
Buckling reduction determination factor	$\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.582**
Buckling reduction factor - eq 6.49	$\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.949**
Design buckling resistance - eq 6.47	$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **1978.1** kN
	$N_{Ed} / N_{b,y,Rd}$ = **0.027**
	PASS - Design buckling resistance exceeds design compression

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

Critical buckling length	$L_{cr,z} = L_{m2\_s1\_seg1}$ = **2300** mm
Critical buckling force	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **6065.9** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **0.586**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_z = **0.49**
Buckling reduction determination factor	$\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **0.766**
Buckling reduction factor - eq 6.49	$\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.793**
Design buckling resistance - eq 6.47	$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **1654.4** kN
	$N_{Ed} / N_{b,z,Rd}$ = **0.033**
	PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m2\_s1\_seg1\_R}$ = **2300** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **102.0** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **7097.8** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **4180** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.706**

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

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_T = **0.49**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.873**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.721**

Design buckling resistance - eq 6.47

$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **1502.9** kN

$N_{Ed} / N_{b,T,Rd} =$ **0.036**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web

$h_w = h - 2 \times t_f =$ **181.2** mm $\eta =$ **1.000**

$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force

$V_{y,Ed} =$ **46.9** kN

Shear area - cl 6.2.6(3)

$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1698** mm²

Design shear resistance - cl 6.2.6(2)

$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **347.9** kN

$V_{y,Ed} / V_{c,y,Rd} =$ **0.135**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$M_{y,Ed} =$ **41.9** kNm

Design bending resistance moment - eq 6.13

$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **176.6** kNm

$M_{y,Ed} / M_{c,y,Rd} =$ **0.237**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$k_c =$ **0.65**

$C_1 = 1 / k_c^2 =$ **2.367**

Poissons ratio

$\nu =$ **0.3**

Shear modulus

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

Unrestrained effective length

$L = 1.0 \times L_{m2\_s1\_seg1\_T} =$ **2300** mm

Elastic critical buckling moment

$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **1584.8** kNm

Slenderness ratio for lateral torsional buckling

$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} =$ **0.334**

Limiting slenderness ratio

$\bar{\lambda}_{LT,0} =$ **0.4**

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

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

$N_{Ed} / N_{y,lim} =$ **0.233**

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

$\psi_y = 41.933$ kNm / -65.866 kNm = **-0.637**

$\alpha_y = -11.966$ kNm / -65.866 kNm = **0.182**

$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4) =$ **0.400**

$\psi_{LT} = 41.933$ kNm / -65.866 kNm = **-0.637**

$\alpha_{LT} = -11.966$ kNm / -65.866 kNm = **0.182**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance

$M_{y,Rk} = W_{pl,y} \times f_y =$ **176.6** kNm

Characteristic moment resistance

$M_{z,Rk} = W_{pl,z} \times f_y =$ **82** kNm

Characteristic resistance to normal force

$N_{Rk} = A \times f_y =$ **2085** kN

Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.402**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.987**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.123**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.267**
	PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f =$ **181.2** mm $\quad \eta =$ **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed} =$ **46.9** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1698** mm^2
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd} =$ **0.135**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed} =$ **65.9** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd} =$ **0.373**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	$k_c =$ **0.65**
	$C_1 = 1 / k_c^2 =$ **2.367**
Poissons ratio	$\nu =$ **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)] =$ **80769** N/mm^2
Unrestrained effective length	$L = 1.0 \times L_{m2\_s1\_seg1\_B} =$ **2300** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **1584.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} =$ **0.334**
Limiting slenderness ratio	$\bar{\lambda}_{LT,0} =$ **0.4**
	$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3	$\psi_y =$ 41.933 kNm / -65.866 kNm = **-0.637**
	$\alpha_y =$ -11.966 kNm / -65.866 kNm = **0.182**
	$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4) =$ **0.400**
	$\psi_{LT} =$ 41.933 kNm / -65.866 kNm = **-0.637**
	$\alpha_{LT} =$ -11.966 kNm / -65.866 kNm = **0.182**
	$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y =$ **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y =$ **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y =$ **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.402**

	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.988**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.176**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.4**
	PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

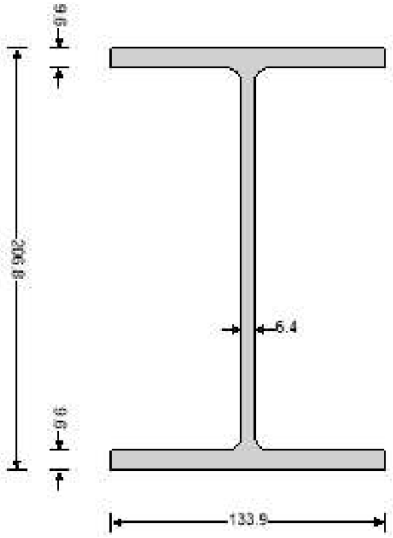
Check y-y axis deflection - Section 7.2.1

Maximum deflection	$\delta_y =$ **4.7** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m2\_s1} / 100 =$ **23** mm
	$\delta_y / \delta_{y,Allowable} =$ **0.205**
	PASS - Allowable deflection exceeds design deflection

3 design

Section details

Section type	UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_f, t_w) =$ **9.6** mm
Nominal yield strength	$f_y =$ **275** N/mm²
Nominal ultimate tensile strength	$f_u =$ **410** N/mm²
Modulus of elasticity	E = **210000** N/mm²



UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 206.8 mm
Section breadth, b, 133.9 mm
Flange thickness, t_f, 9.6 mm
Web thickness, t_w, 6.4 mm
Root radius, r, 7.6 mm
Area of section, A, 3821 mm²
Radius of gyration about y-axis, i_y, 87.051 mm
Radius of gyration about z-axis, i_z, 31.728 mm
Elastic section modulus about y-axis, $W_{el,y}$, 280036 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 57454 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 314366 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 88224 mm³
Second moment of area about y-axis, I_y, 28955684 mm⁴
Second moment of area about z-axis, I_z, 3846542 mm⁴

Lateral restraint

Upper flange has full lateral restraint

Lower flange has lateral restraint at supports plus 1200 & 6000 mm

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} =$ **0.92**

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

Width of section	c = d = **172.4** mm
	$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.597**

$c / t_w = 26.9 = 29.1 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **56.1** mm

$c / t_f = 5.8 = 6.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force $N_{Ed} =$ **58.8** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **1050.8** kN

$N_{Ed} / N_{c,Rd} =$ **0.056**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length $L_{cr,y} = L_{m3\_s1} =$ **6210** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **1556.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.822**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_y =$ **0.21**

Buckling reduction determination factor $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.903**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.783**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **822.9** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.071**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length $L_{cr,T} = L_{m3\_s1\_seg1\_R} =$ **1200** mm

Distance from shear centre to centroid in y axis $y_0 =$ **0.0** mm

Distance from shear centre to centroid in z axis $z_0 =$ **0.0** mm

Radius of gyration $i_0 = \sqrt{(i_y^2 + i_z^2)} =$ **92.7** mm

Elastic critical torsional buckling force $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2 =$ **7239.5** kN

Torsion factor $\beta_T = 1 - (y_0 / i_0)^2 =$ **1**

Elastic critical torsional-flexural buckling force $N_{cr,TF} =$ **4180** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **4180** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})} =$ **0.501**

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

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 $\alpha_T =$ **0.34**

Buckling reduction determination factor $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) =$ **0.677**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1) =$ **0.884**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **928.5** kN

$N_{Ed} / N_{b,T,Rd} =$ **0.063**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f =$ **187.6** mm $\eta =$ **1.000**

$h_w / t_w = 29.3 = 31.7 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{y,Ed}$ = **38.7** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1458** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **231.4** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.167**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **65.9** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **86.5** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.762**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.823**
	$C_1 = 1 / k_c^2$ = **1.478**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m3\_s1\_seg1\_B}$ = **1200** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **866.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.316**
Limiting slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.4**
	$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check bending and axial force - Section 6.2.9

Bending and axial force check - eq.6.33 & eq.6.34	$N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **165.1** kN
	$N_{Ed} / N_{y,lim}$ = **0.356**

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	ψ_y = 13.09 kNm / -65.866 kNm = **-0.199**
	α_y = 13.982 kNm / -65.866 kNm = **-0.212**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = -25.433 kNm / -65.866 kNm = **0.386**
	α_{LT} = -44.142 kNm / -65.866 kNm = **0.670**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.736**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **86.5** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **24.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **1050.8** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.418**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.991**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.39**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.826**
	PASS - Combined bending and compression checks are satisfied

Check design 4623 mm along span

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **23.6** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **86.5** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.273**

PASS - Design bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3	ψ_y = 13.09 kNm / -65.866 kNm = **-0.199**
	α_y = 13.982 kNm / -65.866 kNm = **-0.212**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 15.693 kNm / -25.433 kNm = **-0.617**
	α_{LT} = 19.251 kNm / -25.433 kNm = **-0.757**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **86.5** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **24.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **1050.8** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.415**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.977**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.172**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.325**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design 5668 mm along span

Check y-y axis deflection - Section 7.2.1

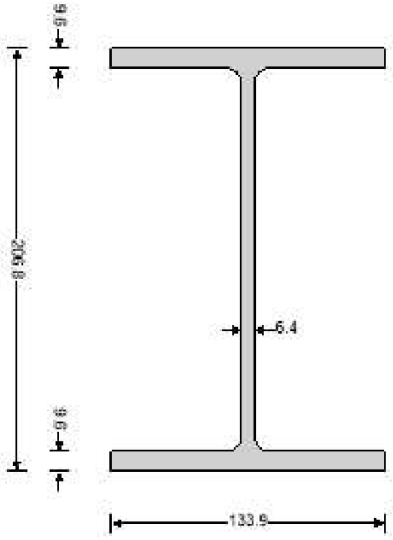
Maximum deflection	δ_y = **28.7** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m3\_s1} / 100$ = **62.1** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.462**

PASS - Allowable deflection exceeds design deflection

4 design

Section details

Section type	UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **9.6** mm
Nominal yield strength	f_y = **275** N/mm^2
Nominal ultimate tensile strength	f_u = **410** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



UB 203x133x30 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 205.8 mm
Section breadth, b, 133.9 mm
Flange thickness, t_f, 9.6 mm
Web thickness, t_w, 6.4 mm
Root radius, r, 7.6 mm
Area of section, A, 3821 mm²
Radius of gyration about y-axis, i_y, 87.051 mm
Radius of gyration about z-axis, i_z, 31.728 mm
Elastic section modulus about y-axis, $W_{el,y}$, 280036 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 57454 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 314365 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 88224 mm³
Second moment of area about y-axis, I_y, 28955684 mm⁴
Second moment of area about z-axis, I_z, 3846542 mm⁴

Lateral restraint

Upper flange has full lateral restraint

Lower flange has lateral restraint at supports plus 1200 & 6000 mm

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ₀S + 1.5ψ₀W (Strength)

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} =$ **0.92**

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

Width of section

$c = d =$ **172.4** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.598**

$c / t_w = 26.9 = 29.1 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$c = (b - t_w - 2 \times r) / 2 =$ **56.1** mm

$c / t_f = 5.8 = 6.3 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force

$N_{Ed} =$ **47** kN

Design resistance of section - eq 6.10

$N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **1050.8** kN

$N_{Ed} / N_{c,Rd} =$ **0.045**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length

$L_{cr,y} = L_{m4\_s1} =$ **6210** mm

Critical buckling force

$N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **1556.4** kN

Slenderness ratio for buckling - eq 6.50

$\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.822**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1

$\alpha_y =$ **0.21**

Buckling reduction determination factor

$\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.903**

Buckling reduction factor - eq 6.49

$\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.783**

Design buckling resistance - eq 6.47

$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **822.9** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.057**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m4\_s1\_seg1\_R}$ = **1200** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **92.7** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **7239.5** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **4180** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.501**

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

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	α_T = **0.34**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.677**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.884**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **928.5** kN
	$N_{Ed} / N_{b,T,Rd}$ = **0.051**

PASS - Design buckling resistance exceeds design compression

Check design 1396 mm along span

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **19.4** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **86.5** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.225**

PASS - Design bending 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	$N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **165.1** kN
	$N_{Ed} / N_{y,lim}$ = **0.301**

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	ψ_y = 12.131 kNm / -67.339 kNm = **-0.180**
	α_y = 8.49 kNm / -67.339 kNm = **-0.126**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 19.282 kNm / -59.945 kNm = **-0.322**
	α_{LT} = 1.235 kNm / -59.945 kNm = **-0.021**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **86.5** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **24.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **1050.8** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.415**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.977**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.154**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.279**

PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **187.6** mm η = **1.000**

$h_w / t_w = 29.3 = 31.7 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed}$ = **36** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1458** mm²

Design shear resistance - cl 6.2.6(2) — $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **231.4** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.156**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed}$ = **67.3** kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **86.5** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.779**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.97**

$C_1 = 1 / k_c^2$ = **1.064**

Poissons ratio — ν = **0.3**

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

Unrestrained effective length — $L = 1.0 \times L_{m4\_s1\_seg3\_B}$ = **210** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **19063.4** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.067**

Limiting slenderness ratio — $\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 — ψ_y = 12.131 kNm / -67.339 kNm = **-0.180**

α_y = 8.49 kNm / -67.339 kNm = **-0.126**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**

ψ_{LT} = -59.945 kNm / -67.339 kNm = **0.890**

α_{LT} = -63.601 kNm / -67.339 kNm = **0.944**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.956**

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

Characteristic moment resistance — $M_{y,Rk} = W_{pl,y} \times f_y$ = **86.5** kNm

Characteristic moment resistance — $M_{z,Rk} = W_{pl,z} \times f_y$ = **24.3** kNm

Characteristic resistance to normal force — $N_{Rk} = A \times f_y$ = **1050.8** kN

Interaction factors — $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.418**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.994**

Interaction formulae - eq 6.61 & eq 6.62 — $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.398**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.846**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection — δ_y = **24.1** mm

Allowable deflection — $\delta_{y,Allowable} = L_{m4\_s1} / 100$ = **62.1** mm

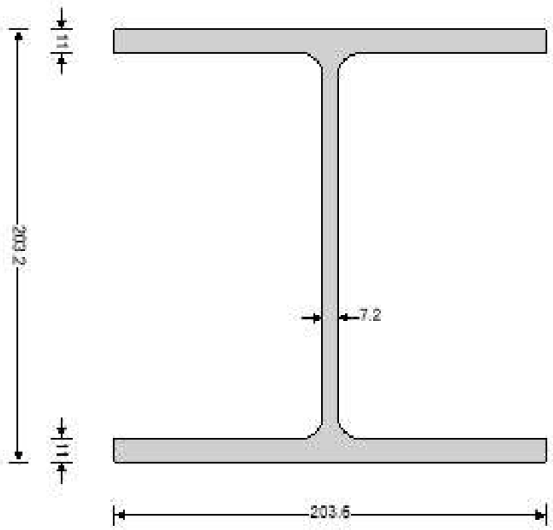
$\delta_y / \delta_{y,Allowable}$ = **0.388**

PASS - Allowable deflection exceeds design deflection

Member5 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **11** mm
Nominal yield strength	f_y = **355** N/mm²
Nominal ultimate tensile strength	f_u = **470** N/mm²
Modulus of elasticity	E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, i_y, 88.19 mm
Radius of gyration about z-axis, i_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230884 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Right flange has full lateral restraint

Left flange has lateral restraint at supports only

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section — c = d = **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.563**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 =$ **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force — $N_{Ed} =$ **50.1** kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **2085** kN

$N_{Ed} / N_{c,Rd} =$ **0.024**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length — $L_{cr,y} = L_{m5\_s1} =$ **2300** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **17896.7** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.341**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — b

Imperfection factor - Table 6.1 — $\alpha_y =$ **0.34**

Buckling reduction determination factor — $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.582**

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.949**

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **1978.1** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.025**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length — $L_{cr,T} = L_{m5\_s1\_seg1\_R} =$ **2300** mm

Distance from shear centre to centroid in y axis — $y_0 =$ **0.0** mm

Distance from shear centre to centroid in z axis — $z_0 =$ **0.0** mm

Radius of gyration — $i_0 = \sqrt{(i_y^2 + i_z^2)} =$ **102.0** mm

Elastic critical torsional buckling force — $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2 =$ **7097.8** kN

Torsion factor — $\beta_T = 1 - (y_0 / i_0)^2 =$ **1**

Elastic critical torsional-flexural buckling force — $N_{cr,TF} =$ **4180** kN

Elastic critical buckling force — $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **4180** kN

Slenderness ratio for torsional buckling - eq 6.52 — $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})} =$ **0.706**

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 — $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) =$ **0.873**

Buckling reduction factor - eq 6.49 — $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1) =$ **0.721**

Design buckling resistance - eq 6.47 — $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **1502.9** kN

$N_{Ed} / N_{b,T,Rd} =$ **0.033**

PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f =$ **181.2** mm $\eta =$ **1.000**

$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{y,Ed}$ = **48.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.138**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **67.3** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.381**
	PASS - Design bending 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
	$N_{Ed} / N_{y,lim}$ = **0.216**
	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	ψ_y = 43.2 kNm / -67.339 kNm = **-0.642**
	α_y = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4)$ = **0.400**
	ψ_{LT} = 43.2 kNm / -67.339 kNm = **-0.642**
	α_{LT} = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** 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**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.988**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.178**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.407**
	PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm η = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **48.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.138**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **43.2** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm

$M_{y,Ed}$ / $M_{c,y,Rd}$ = **0.245**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.65**
	$C_1 = 1 / k_c^2$ = **2.367**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m5\_s1\_seg1\_T}$ = **2300** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **1584.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.334**
Limiting slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.4**

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3	ψ_y = 43.2 kNm / -67.339 kNm = **-0.642**
	α_y = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4)$ = **0.400**
	ψ_{LT} = 43.2 kNm / -67.339 kNm = **-0.642**
	α_{LT} = -12.069 kNm / -67.339 kNm = **0.179**
	$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl.y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl.z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** 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**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.988**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.124**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.273**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection	δ_y = **15.2** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m5\_s1} / 100$ = **23** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.662**

PASS - Allowable deflection exceeds design deflection

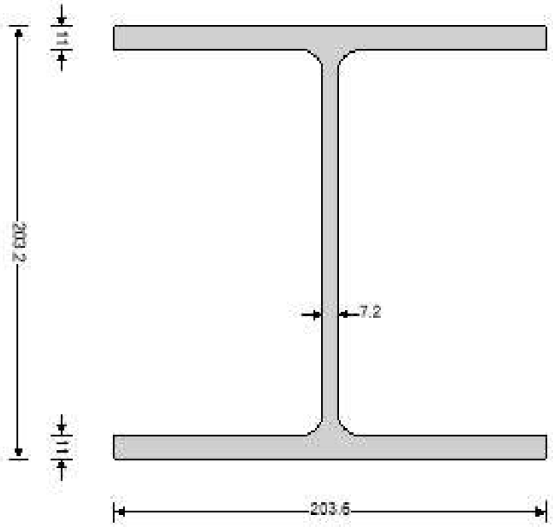
Member6 design

Section details

Section type	UKC 203x203x46 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **11** mm
Nominal yield strength	f_y = **355** N/mm²

Nominal ultimate tensile strength f_u = **470** N/mm²

Modulus of elasticity E = **210000** N/mm²



UKC 203x203x46 (Tata Steel Advance)
Section depth, h, 203.2 mm
Section breadth, b, 203.6 mm
Flange thickness, t_f, 11 mm
Web thickness, t_w, 7.2 mm
Root radius, r, 10.2 mm
Area of section, A, 5873 mm²
Radius of gyration about y-axis, i_y, 88.19 mm
Radius of gyration about z-axis, i_z, 51.343 mm
Elastic section modulus about y-axis, $W_{el,y}$, 449588 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 152083 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 497439 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 230864 mm³
Second moment of area about y-axis, I_y, 45678168 mm⁴
Second moment of area about z-axis, I_z, 15482057 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section c = d = **160.8** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.569**

$c / t_w = 22.3 = 27.4 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 8 = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check compression - Section 6.2.4

Design compression force N_{Ed} = **54.6** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = A \times f_y / \gamma_{M0}$ = **2085** kN

$N_{Ed} / N_{c,Rd}$ = **0.026**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length $L_{cr,y} = L_{m6\_s1}$ = **3400** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **8189.7** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.505**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 α_y = **0.34**

Buckling reduction determination factor $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.679**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.882**

Design buckling resistance - eq 6.47	$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **1839.3** kN
	$N_{Ed} / N_{b,y,Rd}$ = **0.03**
	PASS - Design buckling resistance exceeds design compression

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

Critical buckling length	$L_{cr,z} = L_{m6\_s1\_seg1}$ = **3400** mm
Critical buckling force	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **2775.8** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **0.867**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_z = **0.49**
Buckling reduction determination factor	$\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **1.039**
Buckling reduction factor - eq 6.49	$\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.62**
Design buckling resistance - eq 6.47	$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **1293.6** kN
	$N_{Ed} / N_{b,z,Rd}$ = **0.042**
	PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m6\_s1\_seg1\_R}$ = **3400** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **102.0** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **4180** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **4180** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **4180** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.706**

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

Buckling curve - Table 6.2	c
Imperfection factor - Table 6.1	α_T = **0.49**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.873**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.721**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **1502.9** kN
	$N_{Ed} / N_{b,T,Rd}$ = **0.036**
	PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm $\quad \eta$ = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **13.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.038**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed}$ = **31.2** kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.177**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.676**

$C_1 = 1 / k_c^2$ = **2.19**

Poissons ratio — ν = **0.3**

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

Unrestrained effective length — $L = 1.0 \times L_{m6\_s1\_seg1\_T}$ = **3400** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **761.3** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.482**

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 — α_{LT} = **0.34**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.601**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.968**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.871**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**

Design buckling resistance moment - eq 6.55 — $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.177**

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

$N_{Ed} / N_{y,lim}$ = **0.236**

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 — ψ_y = -13.355 kNm / 31.171 kNm = **-0.428**

α_y = 8.908 kNm / 31.171 kNm = **0.286**

$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4)$ = **0.429**

ψ_{LT} = -13.355 kNm / 31.171 kNm = **-0.428**

α_{LT} = 8.908 kNm / 31.171 kNm = **0.286**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4)$ = **0.429**

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

Characteristic moment resistance — $M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm

Characteristic moment resistance — $M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm

Characteristic resistance to normal force — $N_{Rk} = A \times f_y$ = **2085** kN

Interaction factors — $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.432**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.980**

Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.109**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.221**
	PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **181.2** mm η = **1.000**
	$h_w / t_w = 25.2 = 30.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **13.1** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **347.9** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.038**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **13.4** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **176.6** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.076**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.676**
	$C_1 = 1 / k_c^2$ = **2.19**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m6\_s1\_seg1\_B}$ = **3400** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **761.3** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.482**
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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.601**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.968**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.871**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **176.6** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.076**
	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	ψ_y = -13.355 kNm / 31.171 kNm = **-0.428**
	α_y = 8.908 kNm / 31.171 kNm = **0.286**

C_{my} = max(0.6 + 0.4 × ψ_y, 0.4) = **0.429**

ψ_{LT} = -13.355 kNm / 31.171 kNm = **-0.428**

α_{LT} = 8.908 kNm / 31.171 kNm = **0.286**

C_{mLT} = max(0.6 + 0.4 × ψ_{LT}, 0.4) = **0.429**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **176.6** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **82** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2085** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.433**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.979**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.065**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.12**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection	δ_y = **5.1** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m6\_s1} / 100$ = **34** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.149**

PASS - Allowable deflection exceeds design deflection

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Our ref: 07229E_sH_SE_RP_004

Dr Sally Murrall-Smith PhD
Totnes Renewable Energy Society
107 High Street
Totnes
TQ9 5SN

21st January 2026

Dear Sally,

Structural Check at Dart Marine Park, Steamer Quay Road, Totnes, TQ9 5AL – Building D

As requested, and commissioned, we have carried out a structural survey of the portal frame building at Dart Marine Park, carried out calculations and prepared this report, for the purpose of justifying the proposed PV panel installation to the roof.

The original building is of a full height portalised steel frame, there is an internal mezzanine across a large majority of the building, but this doesn't affect the capacity checks as it's not entirely across the building and could be removed at a later date.

An element of the building is cantilevering over the river path, due to the finishes of the building we could not measure the steels in this area, we requested 'As-Built' drawings, but these haven't been available. Therefore, we can not justify this area on the building.

The portal steel frame has 305 x 165 x 40 UB Rafters and 406 x 178 x 54 UB Columns, the portal frames are at 5.7 metre centres. The dimension of the portal frame are shown in the calculations within this report.

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The proposed PV system is fixed to the roof, loadings are taken as 0.15kN/m^2 which is applied on top of the snow loading at 0.6kN/m^2 and the dead weight loading of 0.5kN/m^2 including services.

Structural calculations have been carried out to check the capacity of these members, and determine adequacy of the existing structure for the proposed PV panel instillation.

We are pleased to confirm that the steel frame member sizes are adequate to support the additional vertical loading of the proposed installation.

Therefore, we can confirm that installing the new roof panels upon the existing roof of this structure will not have a structurally detrimental effect.

EXCEPT THE CANTILEVER AREA AS BELOW: -



Red box shown above in the no PV zone

This letter confirms cover under our PI should any failure of the steel frame occur due to inadequate capacity.

This letter is not intended to and does not relate to the adequacy or assessment of the fixing system to the Photovoltaics to the roof.

Yours sincerely,



Mark Harris MIStructE
Director
structureHaus Ltd

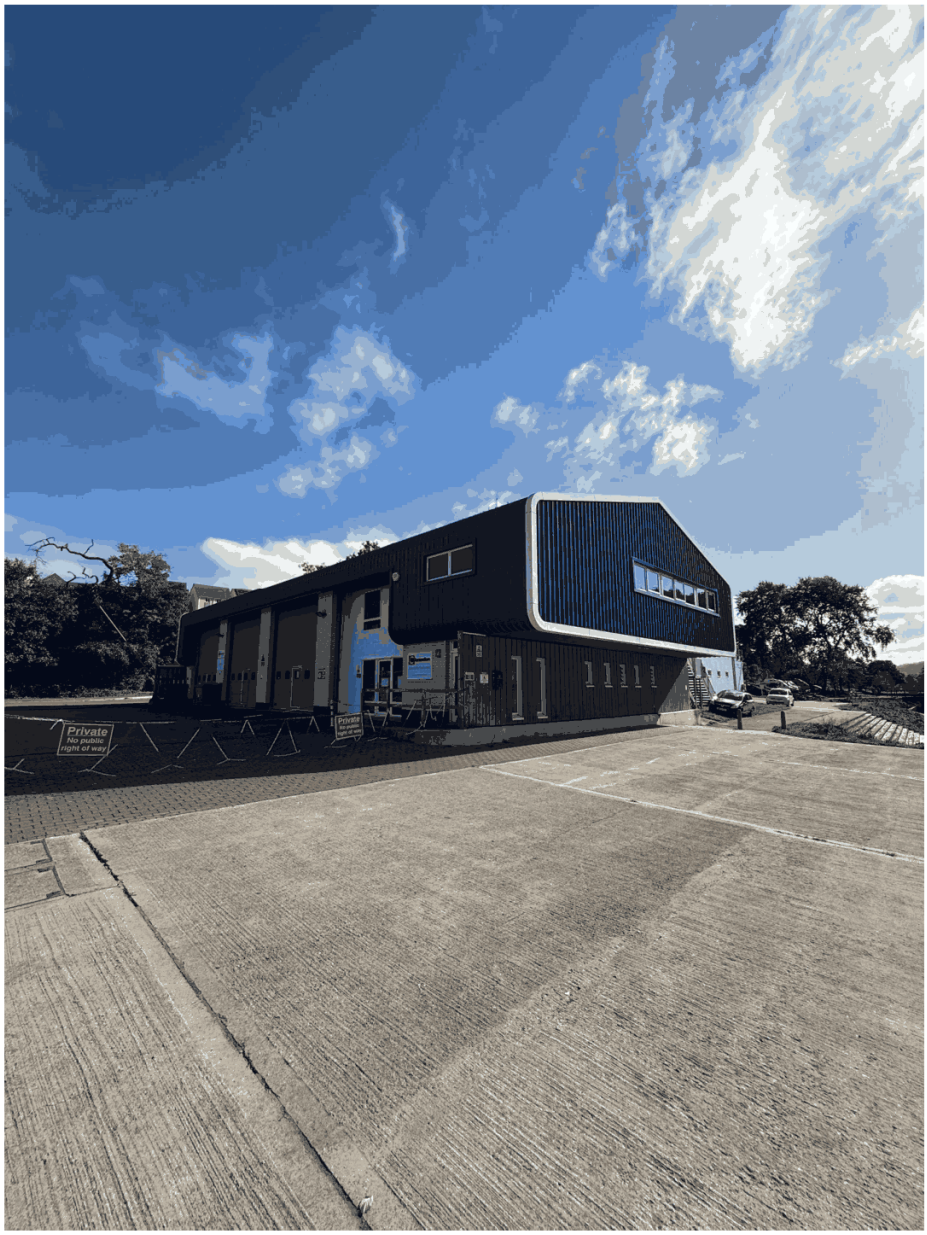
Appendix A – Information Provided and Photos

Appendix B – Structural Calculations

APPENDIX A – INFORMATION PROVIDED & SITE PHOTOS



Proposed PV layout

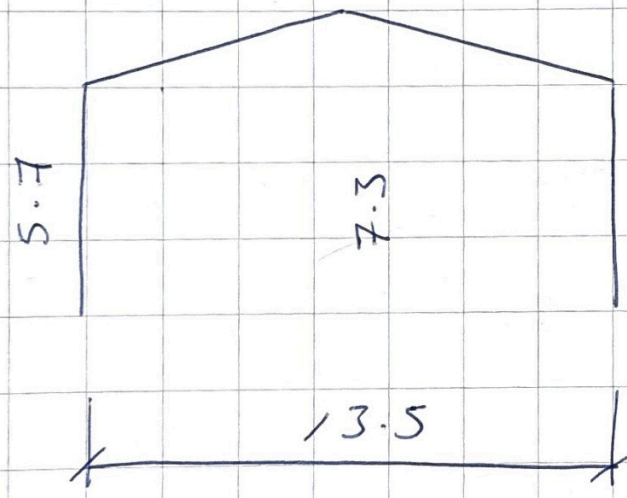


Site Visit Photo

APPENDIX B – STRUCTURAL CALCULATIONS

Building D

This one is disimilar to the others and is a full height portal frame as below:-



@ 5.7 c/c

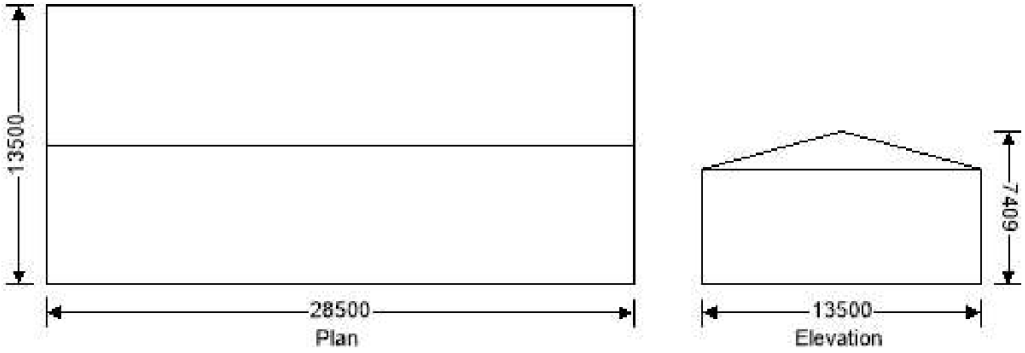
Rafters 305 × 165 × 40 UB

Columns 406 × 178 × 54 UB

WIND LOADING (BS6399)

In accordance with BS6399

Tedds calculation version 3.0.19



Building data

Type of roof	Duopitch
Length of building	L = **28500** mm
Width of building	W = **13500** mm
Height to eaves	H = **5600** mm
Pitch of roof	α_0 = **15.0** deg
Reference height	H_r = **7409** mm

Dynamic classification

Building type factor (Table 1)	K_b = **1.0**
Dynamic augmentation factor (1.6.1)	$C_r = [K_b \times (H_r / (0.1\ m))^{0.75}] / (800 \times \log(H_r / (0.1\ m))) =$ **0.02**

Site wind speed

Location	Totnes
Basic wind speed (Figure 6 BS6399:Pt 2)	V_b = **22.5** m/s
Site altitude	Δ_S = **1** m
Upwind distance from sea to site	d_{sea} = **10** km
Direction factor	S_d =**0.82**
Seasonal factor	S_s = **1.00**
Probability factor	S_p = **1.00**
Critical gap between buildings	g = **5000** mm
Topography not significant	
Altitude factor	$S_a = 1 + 0.001 \times \Delta_S / 1m =$ **1.00**
Site wind speed	$V_s = V_b \times S_a \times S_d \times S_s \times S_p =$ **18.5** m/s
Terrain category	Sea
Displacement height (sheltering effect excluded)	H_d = 0mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

Dynamic pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought)	H_{ref} = **5600**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref}) =$ **5600**mm
Fetch factor (Table 22)	S_c = **0.959**

Turbulence factor (Table 22)	S_t = **0.190**
Gust peak factor	g_t = **3.44**
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.59**
Effective wind speed	$V_e = V_s \times S_b$ = **29.3** m/s
Dynamic pressure	q_s = 0.613 kg/m^3 × V_e^2 = **0.527** kN/m^2

Dynamic pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought)	H_{ref} = **7409**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **7409**mm
Fetch factor (Table 22)	S_c = **1.005**
Turbulence factor (Table 22)	S_t = **0.185**
Gust peak factor	g_t = **3.44**
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.64**
Effective wind speed	$V_e = V_s \times S_b$ = **30.4** m/s
Dynamic pressure	q_s = 0.613 kg/m^3 × V_e^2 = **0.566** kN/m^2

Size effect factors

Diagonal dimension for gablewall	a_{eg} = **14.6** m
External size effect factor gablewall	C_{aeg} = **0.903**
Diagonal dimension for side wall	a_{es} = **29.0** m
External size effect factor side wall	C_{aes} = **0.841**
Diagonal dimension for roof	a_{er} = **29.3** m
External size effect factor roof	C_{aer} = **0.840**
Room/storey volume for internal size effect factor	V_i = **1740.000** m^3
Diagonal dimension for internal size effect factors	$a_i = 10 \times (V_i)^{1/3}$ = **120.277** m
Internal size effect factor	C_{ai} = **0.712**

Pressures and forces

Net pressure	$p = q_s \times c_{pe} \times C_{ae} - q_s \times c_{pi} \times C_{ai}$
Net force	$F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.10	0.57	0.840	-0.60	22.73	-13.71
B (-ve)	-0.80	0.57	0.840	-0.46	20.99	-9.67
C (-ve)	-0.40	0.57	0.840	-0.27	155.44	-42.06
E (-ve)	-1.30	0.57	0.840	-0.70	22.73	-15.87
F (-ve)	-0.90	0.57	0.840	-0.51	20.99	-10.66
G (-ve)	-0.50	0.57	0.840	-0.32	155.44	-49.44

Total vertical net force	$F_{w,v}$ = **-136.59** kN
Total horizontal net force	$F_{w,h}$ = **2.73** kN

Walls load case 1 - Wind 0, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.40	0.57	0.903	-0.80	17.77	-14.19
B	-0.83	0.57	0.903	-0.51	70.04	-35.51
w	0.73	0.53	0.841	0.25	159.60	39.78
l	-0.50	0.53	0.841	-0.30	159.60	-47.29

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-47.3** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **39.8** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **77.6** kN

Roof load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A (+ve)	0.20	0.57	0.840	0.22	22.73	4.91
B (+ve)	0.20	0.57	0.840	0.22	20.99	4.53
C (+ve)	0.20	0.57	0.840	0.22	155.44	33.55
E (+ve)	-1.30	0.57	0.840	-0.50	22.73	-11.29
F (+ve)	-0.90	0.57	0.840	-0.31	20.99	-6.44
G (+ve)	-0.50	0.57	0.840	-0.12	155.44	-18.14

Total vertical net force $F_{w,v}$ = **6.87** kN

Total horizontal net force $F_{w,h}$ = **20.41** kN

Walls load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.40	0.57	0.903	-0.60	17.77	-10.61
B	-0.83	0.57	0.903	-0.31	70.04	-21.41
w	0.73	0.53	0.841	0.44	159.60	69.70
l	-0.50	0.53	0.841	-0.11	159.60	-17.38

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-17.4** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **69.7** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **92.9** kN

Roof load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.60	0.57	0.840	-0.84	9.43	-7.93
B (-ve)	-1.50	0.57	0.840	-0.79	9.43	-7.48
C (-ve)	-0.60	0.57	0.840	-0.37	75.47	-27.59
D (-ve)	-0.40	0.57	0.840	-0.27	303.98	-82.25

Total vertical net force $F_{w,v}$ = **-120.99** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.54	0.53	0.841	-0.75	12.54	-9.47
B	-0.88	0.53	0.841	-0.46	50.18	-23.28
C	-0.81	0.53	0.841	-0.44	96.88	-42.19
w	0.61	0.57	0.903	0.23	87.81	20.41
l	-0.50	0.57	0.903	-0.34	87.81	-29.50

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-29.5** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **20.4** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **43.1** kN

Roof load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.60	0.57	0.840	-0.64	9.43	-6.03
B (-ve)	-1.50	0.57	0.840	-0.59	9.43	-5.58
C (-ve)	-0.60	0.57	0.840	-0.16	75.47	-12.39
D (-ve)	-0.40	0.57	0.840	-0.07	303.98	-21.04

Total vertical net force $F_{w,v}$ = **-43.51** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

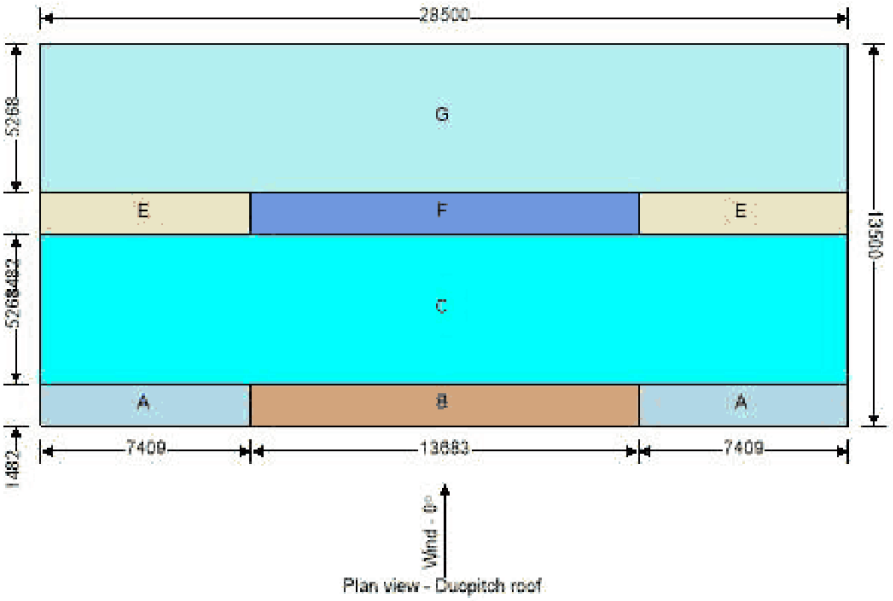
Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m²)	External size factor, C_{ae}	Net Pressure, p (kN/m²)	Area, A_{ref} (m²)	Net force, F_w (kN)
A	-1.54	0.53	0.841	-0.57	12.54	-7.12
B	-0.88	0.53	0.841	-0.28	50.18	-13.87
C	-0.81	0.53	0.841	-0.25	96.88	-24.03
w	0.61	0.57	0.903	0.43	87.81	38.09
l	-0.50	0.57	0.903	-0.13	87.81	-11.82

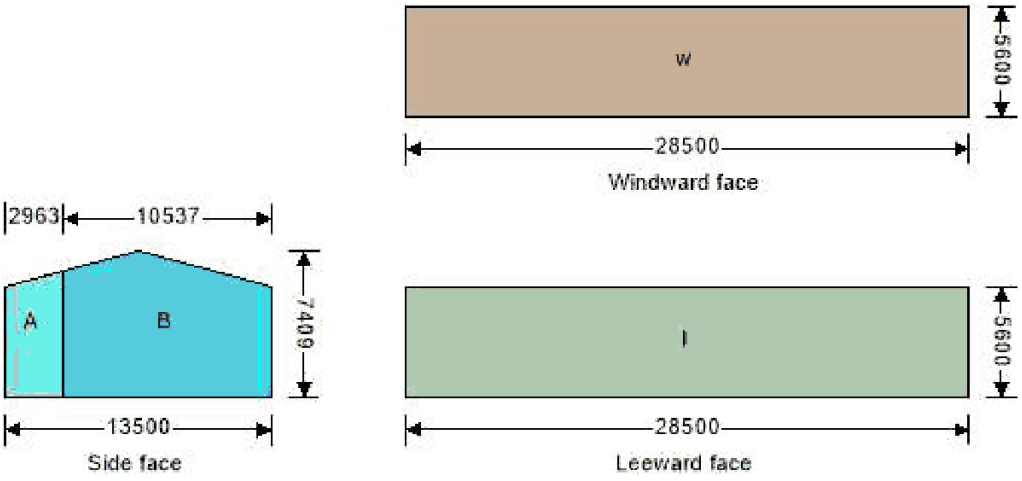
Overall loading

Equiv leeward net force for overall section — $F_l = F_{w,wl}$ = **-11.8** kN

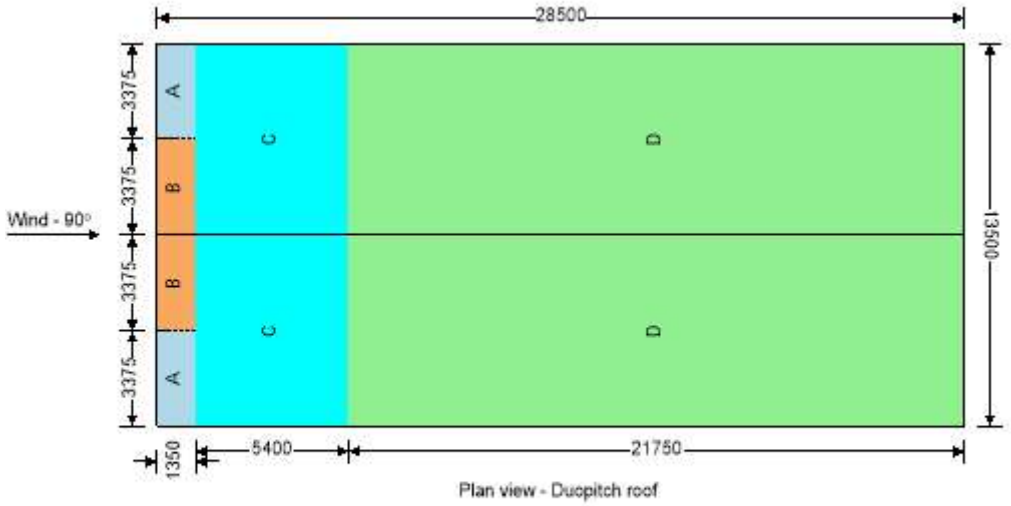
Net windward force for overall section — $F_w = F_{w,ww}$ = **38.1** kN

Overall loading overall section — $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **43.1** kN

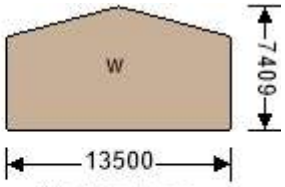




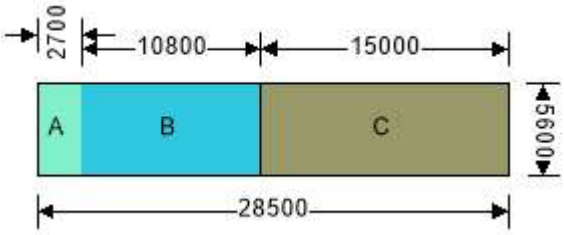
structureHaus Ltd	Project: Dart Marine Park, Totnes				Job no.: 07229E	
	Calcs for: Wind				Start page no./Revision: 7	
	Calcs by: MDH	Calcs date: 11/11/2025	Checked by: DALP	Checked date:	Approved by:	Approved date:



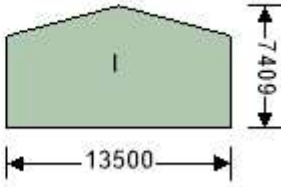
Plan view - Duopitch roof



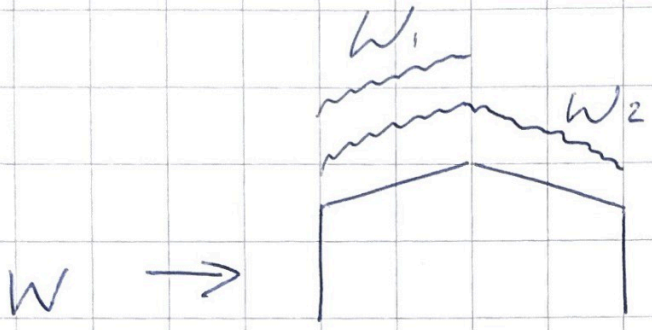
Windward face



Side face



Leeward face



$W = 1.12 \times 5.7 = 6.38\ kN/m$

$W_1 = 0.12 \times 5.7 = 0.68\ kN/m$

$W_2 = 0.5 \times 5.7 = 2.85\ kN/m$

$0.6 = 3.42\ kN/m$

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

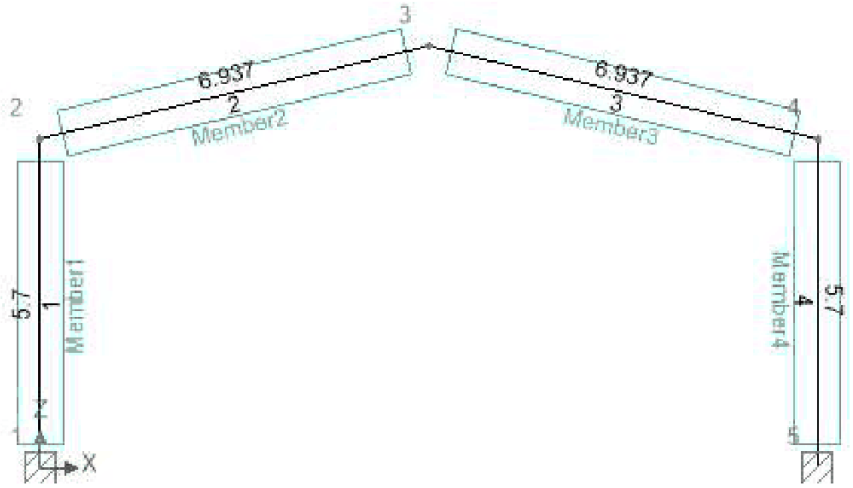
Tedds calculation version 4.5.06

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (m) - Steel (EC3)



Materials

Name	Density (kg/m³)	Youngs Modulus kN/mm²	Shear Modulus kN/mm²	Thermal Coefficient °C⁻¹
Steel (EC3)	7850	210	80.8	0.000012

Sections

Name	Area (cm²)	Moment of inertia		Shear area parallel to	
		Major (cm⁴)	Minor (cm⁴)	Minor (cm²)	Major (cm²)
UB 305x165x40	51.3	8502.6	764.4	18.2	30.3
UB 406x178x54	69.4	18893.3	1021.5	31	34.9

Nodes

Node	Co-ordinates		Freedom			Coordinate system		Spring		
	X (m)	Z (m)	X	Z	Rot.	Name	Angle (°)	X (kN/m)	Z (kN/m)	Rot. kNm/°
1	0	0	Fixed	Fixed	Fixed		0	0	0	0
2	0	5.7	Free	Free	Free		0	0	0	0
3	6.75	7.3	Free	Free	Free		0	0	0	0
4	13.5	5.7	Free	Free	Free		0	0	0	0
5	13.5	0	Fixed	Fixed	Fixed		0	0	0	0

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	5.7	1	2	UB 406x178x54	Steel (EC3)	Fixed	Fixed	Fixed	
2	6.937	2	3	UB 305x165x40	Steel (EC3)	Fixed	Fixed	Fixed	
3	6.937	3	4	UB 305x165x40	Steel (EC3)	Fixed	Fixed	Fixed	
4	5.7	4	5	UB 406x178x54	Steel (EC3)	Fixed	Fixed	Fixed	

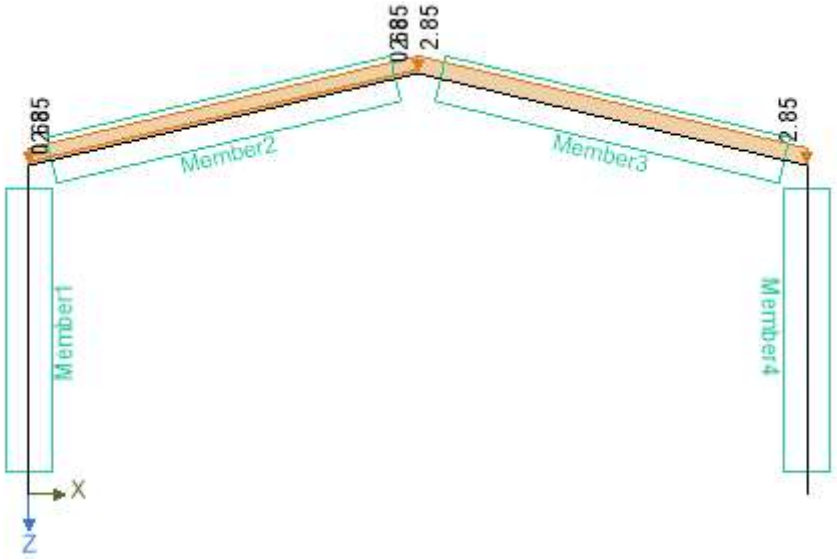
Members

Name	Elements	
	Start	End
Member1	1	1
Member2	2	2
Member3	3	3
Member4	4	4

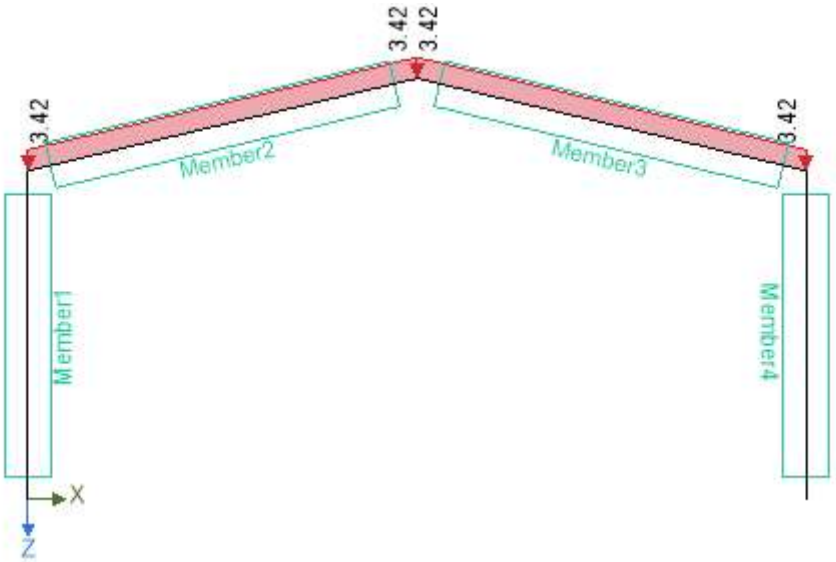
Loading

Self weight included

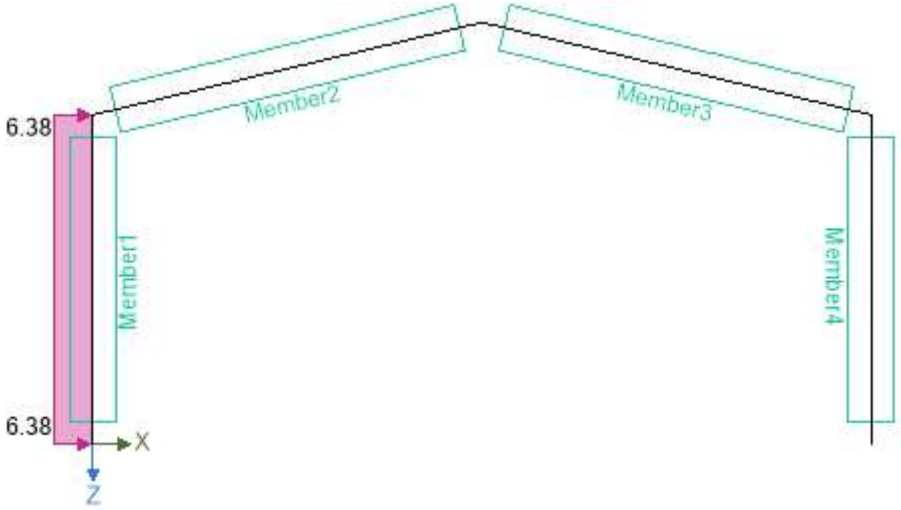
Permanent - Loading (kN/m)



Imposed - Loading (kN/m)



Wind - Loading (kN/m)



Load combination factors

Load combination	Self Weight	Permanent	Imposed	Wind
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50	
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00	
1.35G + 1.5Q + 1.5ψ_0S + 1.5ψ_0W (Strength)	1.35	1.35	1.50	0.75
1.0G + 1.0Q + 0.5S + 0.5W (Service)	1.00	1.00	1.00	0.50

Member Loads

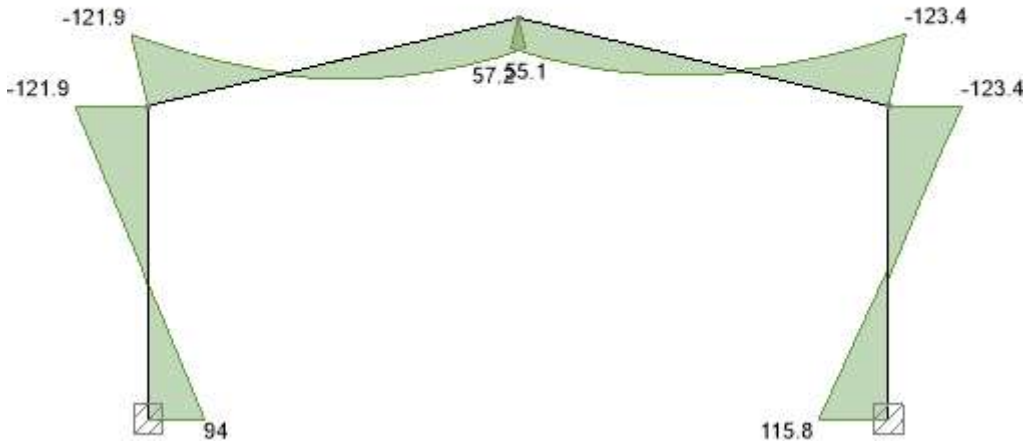
Member	Load case	Load Type	Orientation	Description
Member2	Permanent	UDL	GlobalZ	2.85 kN/m

Member	Load case	Load Type	Orientation	Description
Member2	Permanent	UDL	GlobalZ	0.68 kN/m
Member3	Permanent	UDL	GlobalZ	2.85 kN/m
Member2	Imposed	UDL	GlobalZ	3.42 kN/m
Member3	Imposed	UDL	GlobalZ	3.42 kN/m
Member1	Wind	UDL	GlobalX	6.38 kN/m

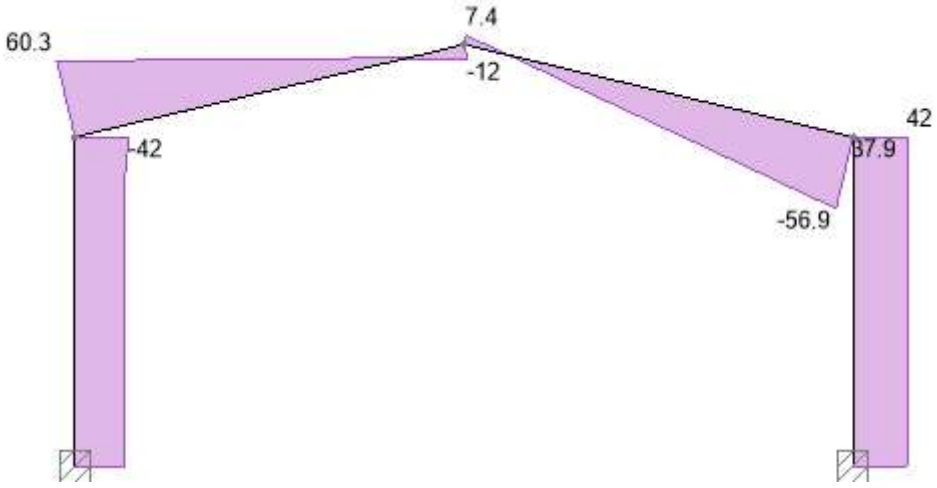
Results

Forces

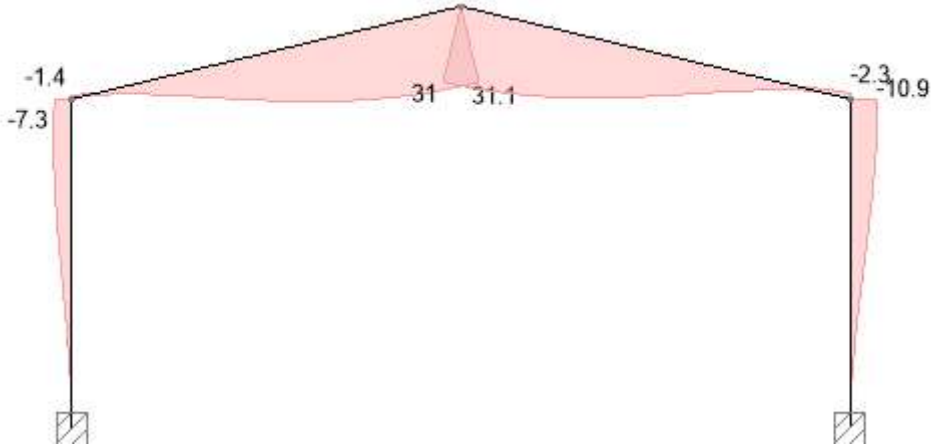
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



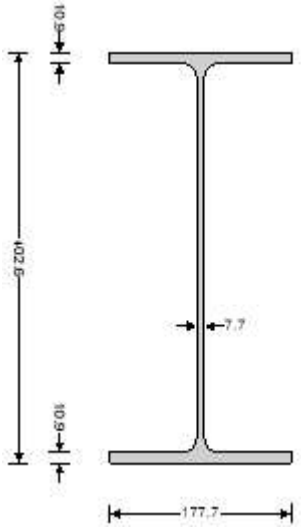
Partial factors - Section 6.1

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**

Member1 design

Section details

Section type	UB 406x178x54 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	t_{nom} = max(t_f, t_w) = **10.9** 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



UB 406x178x54 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 402.6 mm
Section breadth, b, 177.7 mm
Flange thickness, t_f, 10.9 mm
Web thickness, t_w, 7.7 mm
Root radius, r, 12.7 mm
Area of section, A, 6944 mm^2
Radius of gyration about y-axis, i_y, 164.943 mm
Radius of gyration about z-axis, i_z, 38.354 mm
Elastic section modulus about y-axis, $W_{el,y}$, 938563 mm^3
Elastic section modulus about z-axis, $W_{el,z}$, 114972 mm^3
Plastic section modulus about y-axis, $W_{pl,y}$, 1063806 mm^3
Plastic section modulus about z-axis, $W_{pl,z}$, 178666 mm^3
Second moment of area about y-axis, I_y, 188932782 mm^4
Second moment of area about z-axis, I_z, 10215296 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section

$c = d =$ **355.4** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.539**

$c / t_w = 46.2 = 56.7 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$c = (b - t_w - 2 \times r) / 2 =$ **72.3** mm

$c / t_f = 6.6 = 8.2 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force — $N_{Ed} =$ **75.1** kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **2465.3** kN

$N_{Ed} / N_{c,Rd} =$ **0.03**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length — $L_{cr,y} = L_{m1\_s1} =$ **5700** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **12052.5** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.452**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — a

Imperfection factor - Table 6.1 — $\alpha_y =$ **0.21**

Buckling reduction determination factor — $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.629**

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.938**

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **2313.6** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.032**

PASS - Design buckling resistance exceeds design compression

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

Critical buckling length — $L_{cr,z} = L_{m1\_s1\_seg1} =$ **5700** mm

Critical buckling force — $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **651.7** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **1.945**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 — b

Imperfection factor - Table 6.1 — $\alpha_z =$ **0.34**

Buckling reduction determination factor — $\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **2.688**

Buckling reduction factor - eq 6.49 — $\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1) =$ **0.22**

Design buckling resistance - eq 6.47 — $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **542.6** kN

$N_{Ed} / N_{b,z,Rd} =$ **0.138**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length — $L_{cr,T} = L_{m1\_s1\_seg1\_R} =$ **5700** mm

Distance from shear centre to centroid in y axis — $y_0 =$ **0.0** mm

Distance from shear centre to centroid in z axis — $z_0 =$ **0.0** mm

Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)} =$ **169.3** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2 =$ **1557.3** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2 =$ **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF} =$ **1557.3** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **1557.3** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})} =$ **1.258**

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

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	$\alpha_T =$ **0.34**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) =$ **1.471**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1) =$ **0.448**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **1103.4** kN
	$N_{Ed} / N_{b,T,Rd} =$ **0.068**
	PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f =$ **380.8** mm $\quad \eta =$ **1.000**
	$h_w / t_w = 49.5 = 60.8 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed} =$ **37.9** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **3431** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **703.3** kN
	$V_{y,Ed} / V_{c,y,Rd} =$ **0.054**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed} =$ **94** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **377.7** kNm
	$M_{y,Ed} / M_{c,y,Rd} =$ **0.249**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	$k_c =$ **0.65**
	$C_1 = 1 / k_c^2 =$ **2.367**
Poissons ratio	$\nu =$ **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)] =$ **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m1\_s1\_seg1\_T} =$ **5700** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **403.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} =$ **0.967**
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	c
Imperfection factor - Table 6.3	$\alpha_{LT} =$ **0.49**
Correction factor for rolled sections	$\beta =$ **0.75**

LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2) =$ **0.990**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.659**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **0.835**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.790**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} =$ **298.2** kNm
	$M_{y,Ed} / M_{b,y,Rd} =$ **0.315**

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	$N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0}) =$ **520.5** kN
	$N_{Ed} / N_{y,lim} =$ **0.144**

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	$\psi_y = 93.991$ kNm / -121.889 kNm = **-0.771**
	$\alpha_y = -13.949$ kNm / -121.889 kNm = **0.114**
	$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4) =$ **0.400**
	$\psi_{LT} = 93.991$ kNm / -121.889 kNm = **-0.771**
	$\alpha_{LT} = -13.949$ kNm / -121.889 kNm = **0.114**
	$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y =$ **377.7** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y =$ **63.4** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y =$ **2465.3** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.403**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.908**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.185**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.481**

PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f =$ **380.8** mm $\quad \eta =$ **1.000**
	$h_w / t_w = 49.5 = 60.8 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{y,Ed} =$ **37.9** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **3431** mm^2
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **703.3** kN
	$V_{y,Ed} / V_{c,y,Rd} =$ **0.054**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed} =$ **121.9** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **377.7** kNm
	$M_{y,Ed} / M_{c,y,Rd} =$ **0.323**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.65**
	$C_1 = 1 / k_c^2$ = **2.367**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2
Unrestrained effective length	$L = 1.0 \times L_{m1\_s1\_seg1\_B}$ = **5700** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **403.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.967**
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	c
Imperfection factor - Table 6.3	α_{LT} = **0.49**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.990**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.659**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.835**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.790**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **298.2** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.409**
	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	ψ_y = 93.991 kNm / -121.889 kNm = **-0.771**
	α_y = -13.949 kNm / -121.889 kNm = **0.114**
	$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4)$ = **0.400**
	ψ_{LT} = 93.991 kNm / -121.889 kNm = **-0.771**
	α_{LT} = -13.949 kNm / -121.889 kNm = **0.114**
	$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl.y} \times f_y$ = **377.7** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl.z} \times f_y$ = **63.4** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2465.3** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.403**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.913**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.228**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.578**
	PASS - Combined bending and compression checks are satisfied

Check design 5026 mm along span

Check y-y axis deflection - Section 7.2.1

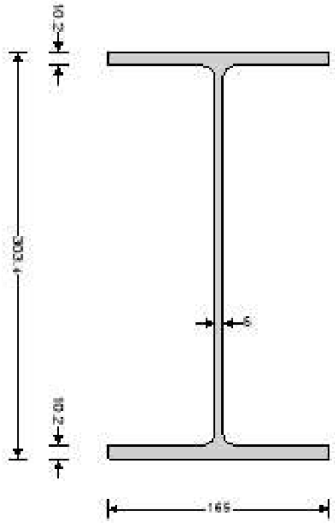
Maximum deflection	δ_y = **10.5** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m1\_s1} / 150$ = **38** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.276**

PASS - Allowable deflection exceeds design deflection

Member2 design

Section details

Section type	UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	t_{nom} = max(t_f, t_w) = **10.2** 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



UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 303.4 mm
Section breadth, b, 165 mm
Flange thickness, t_f, 10.2 mm
Web thickness, t_w, 6 mm
Root radius, r, 8.9 mm
Area of section, A, 5132 mm^2
Radius of gyration about y-axis, i_y, 128.716 mm
Radius of gyration about z-axis, i_z, 38.593 mm
Elastic section modulus about y-axis, $W_{el,y}$, 660488 mm^3
Elastic section modulus about z-axis, $W_{el,z}$, 92650 mm^3
Plastic section modulus about y-axis, $W_{pl,y}$, 823075 mm^3
Plastic section modulus about z-axis, $W_{pl,z}$, 141733 mm^3
Second moment of area about y-axis, I_y, 86026974 mm^4
Second moment of area about z-axis, I_z, 7643988 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section

c = d = **265.2** mm

α = min([h / 2 + N_{Ed} / (2 × t_w × f_y) - (t_f + r)] / c, 1) = **0.547**

c / t_w = 44.2 = 54.3 × ε <= 396 × ε / (13 × α - 1) Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

c = (b - t_w - 2 × r) / 2 = **70.6** mm

c / t_f = 6.9 = 8.5 × ε <= 9 × ε Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force	N_{Ed} = **53.2** kN
Design resistance of section - eq 6.10	$N_{c,Rd}$ = A × f_y / γ_{M0} = **1821.9** kN
	N_{Ed} / $N_{c,Rd}$ = **0.029**

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} = **6937** mm

Critical buckling force	$N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **3662** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.705**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	a
Imperfection factor - Table 6.1	α_y = **0.21**
Buckling reduction determination factor	$\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.802**
Buckling reduction factor - eq 6.49	$\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.845**
Design buckling resistance - eq 6.47	$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **1539.9** kN
	$N_{Ed} / N_{b,y,Rd}$ = **0.035**
	PASS - Design buckling resistance exceeds design compression

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

Critical buckling length	$L_{cr,z} = L_{m2\_s1\_seg1}$ = **6937** mm
Critical buckling force	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2$ = **329.2** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **2.352**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	α_z = **0.34**
Buckling reduction determination factor	$\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2)$ = **3.633**
Buckling reduction factor - eq 6.49	$\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1)$ = **0.156**
Design buckling resistance - eq 6.47	$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **284.6** kN
	$N_{Ed} / N_{b,z,Rd}$ = **0.187**
	PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m2\_s1\_seg1\_R}$ = **6937** mm
Distance from shear centre to centroid in y axis	y_0 = **0.0** mm
Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **134.4** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **1050.9** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **1557.3** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **1050.9** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **1.317**

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

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	α_T = **0.34**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **1.557**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.419**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **763.2** kN
	$N_{Ed} / N_{b,T,Rd}$ = **0.07**
	PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **283** mm η = **1.000**
	$h_w / t_w = 47.2 = 58 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{y,Ed}$ = **60.3** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **2009** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **411.7** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.146**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **121.9** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **221.2** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.551**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.579**
	$C_1 = 1 / k_c^2$ = **2.985**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m2\_s1\_seg1\_B}$ = **6937** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **235.9** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.968**
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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.948**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.719**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.801**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.897**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **198.5** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.614**

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	$N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **301.4** kN
	$N_{Ed} / N_{y,lim}$ = **0.177**

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	ψ_y = 52.198 kNm / -121.889 kNm = **-0.428**
	α_y = 26.196 kNm / -121.889 kNm = **-0.215**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.400**
	ψ_{LT} = 52.198 kNm / -121.889 kNm = **-0.428**
	α_{LT} = 26.196 kNm / -121.889 kNm = **-0.215**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y =$ **221.2** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y =$ **50.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y =$ **1821.9** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.407**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.875**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.346**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.858**
	PASS - Combined bending and compression checks are satisfied

Check design 5942 mm along span

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed} =$ **57.2** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **221.2** kNm
	$M_{y,Ed} / M_{c,y,Rd} =$ **0.259**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	$k_c =$ **0.579**
	$C_1 = 1 / k_c^2 =$ **2.985**
Poissons ratio	$\nu =$ **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)] =$ **80769** N/mm^2
Unrestrained effective length	$L = 1.0 \times L_{m2\_s1\_seg1\_T} =$ **6937** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **235.9** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} =$ **0.968**
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	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2) =$ **0.948**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.719**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **0.801**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.897**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} =$ **198.5** kNm
	$M_{y,Ed} / M_{b,y,Rd} =$ **0.288**
	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	$\psi_y =$ 52.198 kNm / -121.889 kNm = **-0.428**
	$\alpha_y =$ 26.196 kNm / -121.889 kNm = **-0.215**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4) =$ **0.400**
	$\psi_{LT} =$ 52.198 kNm / -121.889 kNm = **-0.428**
	$\alpha_{LT} =$ 26.196 kNm / -121.889 kNm = **-0.215**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl.y} \times f_y =$ **221.2** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl.z} \times f_y =$ **50.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y =$ **1821.9** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.405**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.909**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.171**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.464**

PASS - Combined bending and compression checks are satisfied

Check design 6826 mm along span

Check y-y axis deflection - Section 7.2.1

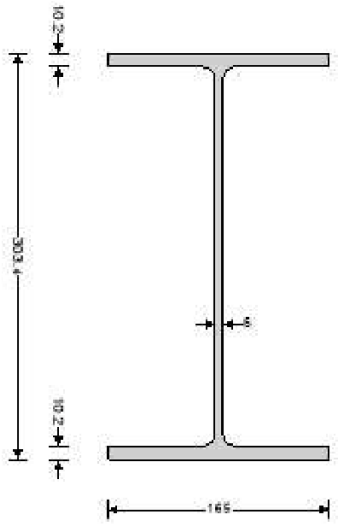
Maximum deflection	$\delta_y =$ **44.3** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m2\_s1} / 150 =$ **46.2** mm
	$\delta_y / \delta_{y,Allowable} =$ **0.958**

PASS - Allowable deflection exceeds design deflection

Member3 design

Section details

Section type	UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w) =$ **10.2** 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



UB 305x165x40 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 303.4 mm
Section breadth, b, 165 mm
Flange thickness, t_f, 10.2 mm
Web thickness, t_w, 6 mm
Root radius, r, 8.9 mm
Area of section, A, 5132 mm^2
Radius of gyration about y-axis, i_y, 128.716 mm
Radius of gyration about z-axis, i_z, 38.593 mm
Elastic section modulus about y-axis, $W_{el,y}$, 560488 mm^3
Elastic section modulus about z-axis, $W_{el,z}$, 92650 mm^3
Plastic section modulus about y-axis, $W_{pl,y}$, 623075 mm^3
Plastic section modulus about z-axis, $W_{pl,z}$, 141733 mm^3
Second moment of area about y-axis, I_y, 86026974 mm^4
Second moment of area about z-axis, I_z, 7643588 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ0S + 1.5ψ0W (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section: $c = d =$ **265.2** mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.550**

$c / t_w = 44.2 = 54.3 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section: $c = (b - t_w - 2 \times r) / 2 =$ **70.6** mm

$c / t_f = 6.9 = 8.5 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force: $N_{Ed} =$ **41.4** kN

Design resistance of section - eq 6.10: $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **1821.9** kN

$N_{Ed} / N_{c,Rd} =$ **0.023**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length: $L_{cr,y} = L_{m3\_s1} =$ **6937** mm

Critical buckling force: $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **3662** kN

Slenderness ratio for buckling - eq 6.50: $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.705**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2: a

Imperfection factor - Table 6.1: $\alpha_y =$ **0.21**

Buckling reduction determination factor: $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.802**

Buckling reduction factor - eq 6.49: $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.845**

Design buckling resistance - eq 6.47: $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **1539.9** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.027**

PASS - Design buckling resistance exceeds design compression

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

Critical buckling length: $L_{cr,z} = L_{m3\_s1\_seg1} =$ **6937** mm

Critical buckling force: $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **329.2** kN

Slenderness ratio for buckling - eq 6.50: $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **2.352**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2: b

Imperfection factor - Table 6.1: $\alpha_z =$ **0.34**

Buckling reduction determination factor: $\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **3.633**

Buckling reduction factor - eq 6.49: $\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1) =$ **0.156**

Design buckling resistance - eq 6.47: $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **284.6** kN

$N_{Ed} / N_{b,z,Rd} =$ **0.145**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length: $L_{cr,T} = L_{m3\_s1\_seg1\_R} =$ **6937** mm

Distance from shear centre to centroid in y axis: $y_0 =$ **0.0** mm

Distance from shear centre to centroid in z axis	z_0 = **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **134.4** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **1050.9** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **1557.3** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **1050.9** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **1.317**

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

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	α_T = **0.34**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **1.557**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.419**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **763.2** kN
	$N_{Ed} / N_{b,T,Rd}$ = **0.054**
	PASS - Design buckling resistance exceeds design compression

Check design 792 mm along span

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **51.3** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **221.2** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.232**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.57**
	$C_1 = 1 / k_c^2$ = **3.077**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2
Unrestrained effective length	$L = 1.0 \times L_{m3\_s1\_seg1\_T}$ = **6937** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **243.2** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.954**
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	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.935**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.728**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.795**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.915**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **202.5** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.253**
	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 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0}) =$ **301.4** kN

$N_{Ed} / N_{y,lim} =$ **0.143**

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

$\psi_y = 48.399 \text{ kNm} / -123.405 \text{ kNm} =$ **-0.392**

$\alpha_y = 18.165 \text{ kNm} / -123.405 \text{ kNm} =$ **-0.147**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4) =$ **0.400**

$\psi_{LT} = 48.399 \text{ kNm} / -123.405 \text{ kNm} =$ **-0.392**

$\alpha_{LT} = 18.165 \text{ kNm} / -123.405 \text{ kNm} =$ **-0.147**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance $M_{y,Rk} = W_{pl,y} \times f_y =$ **221.2** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl,z} \times f_y =$ **50.3** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y =$ **1821.9** kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.406**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.899**

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.157**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.438**

PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f =$ **283** mm $\eta =$ **1.000**

$h_w / t_w = 47.2 = 58 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed} =$ **56.9** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **2009** mm²

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **411.7** kN

$V_{y,Ed} / V_{c,y,Rd} =$ **0.138**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed} =$ **123.4** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **221.2** kNm

$M_{y,Ed} / M_{c,y,Rd} =$ **0.558**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c =$ **0.57**

$C_1 = 1 / k_c^2 =$ **3.077**

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

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

Unrestrained effective length $L = 1.0 \times L_{m3\_s1\_seg1\_B} =$ **6937** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **243.2** kNm

Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **0.954**
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	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2) =$ **0.935**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.728**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) =$ **0.795**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.915**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **202.5** kNm
	$M_{y,Ed} / M_{b,y,Rd} =$ **0.61**

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	$\psi_y =$ 48.399 kNm / -123.405 kNm = **-0.392**
	$\alpha_y =$ 18.165 kNm / -123.405 kNm = **-0.147**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4) =$ **0.400**
	$\psi_{LT} =$ 48.399 kNm / -123.405 kNm = **-0.392**
	$\alpha_{LT} =$ 18.165 kNm / -123.405 kNm = **-0.147**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl.y} \times f_y =$ **221.2** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl.z} \times f_y =$ **50.3** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y =$ **1821.9** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.407**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.867**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.349**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.864**

PASS - Combined bending and compression checks are satisfied

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

Check design at start of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection	$\delta_y =$ **44.1** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m3\_s1} / 150 =$ **46.2** mm
	$\delta_y / \delta_{y,Allowable} =$ **0.953**

PASS - Allowable deflection exceeds design deflection

Member4 design

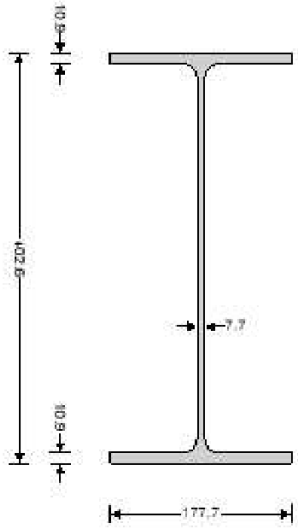
Section details

Section type	UB 406x178x54 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S355
Nominal thickness of element	$t_{nom} = \max(t_f, t_w) =$ **10.9** mm

Nominal yield strength f_y = **355** N/mm²

Nominal ultimate tensile strength f_u = **470** N/mm²

Modulus of elasticity E = **210000** N/mm²



UB 406x178x54 (British Steel Section Range 2022 (B34-1))
Section depth, h, 402.6 mm
Section breadth, b, 177.7 mm
Flange thickness, t_f, 10.9 mm
Web thickness, t_w, 7.7 mm
Root radius, r, 12.7 mm
Area of section, A, 6944 mm²
Radius of gyration about y-axis, i_y, 164.943 mm
Radius of gyration about z-axis, i_z, 38.354 mm
Elastic section modulus about y-axis, $W_{el,y}$, 936563 mm³
Elastic section modulus about z-axis, $W_{el,z}$, 114572 mm³
Plastic section modulus about y-axis, $W_{pl,y}$, 1063806 mm³
Plastic section modulus about z-axis, $W_{pl,z}$, 178666 mm³
Second moment of area about y-axis, I_y, 188932782 mm⁴
Second moment of area about z-axis, I_z, 10215296 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 3 - 1.35G + 1.5Q + 1.5ψ₀S + 1.5ψ₀W (Strength)

Classification of cross sections - Section 5.5

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

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

Width of section c = d = **355.4** mm

α = min([h / 2 + N_{Ed} / (2 × t_w × f_y) - (t_f + r)] / c, 1) = **0.537**

c / t_w = 46.2 = 56.7 × ε <= 396 × ε / (13 × α - 1) Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section c = (b - t_w - 2 × r) / 2 = **72.3** mm

c / t_f = 6.6 = 8.2 × ε <= 9 × ε Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force N_{Ed} = **68.4** kN

Design resistance of section - eq 6.10 $N_{c,Rd}$ = A × f_y / γ_{M0} = **2465.3** kN

N_{Ed} / $N_{c,Rd}$ = **0.028**

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_{m4_s1} = **5700** mm

Critical buckling force $N_{cr,y}$ = π^2 × E × I_y / $L_{cr,y}^2$ = **12052.5** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y$ = $\sqrt{(A \times f_y / N_{cr,y})}$ = **0.452**

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 α_y = **0.21**

Buckling reduction determination factor Φ_y = 0.5 × (1 + α_y × ($\bar{\lambda}_y$ - 0.2) + $\bar{\lambda}_y^2$) = **0.629**

Buckling reduction factor - eq 6.49	$\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.938**
Design buckling resistance - eq 6.47	$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **2313.6** kN
	$N_{Ed} / N_{b,y,Rd} =$ **0.03**
	PASS - Design buckling resistance exceeds design compression

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

Critical buckling length	$L_{cr,z} = L_{m4\_s1\_seg1} =$ **5700** mm
Critical buckling force	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **651.7** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **1.945**

Check z-z axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	$\alpha_z =$ **0.34**
Buckling reduction determination factor	$\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **2.688**
Buckling reduction factor - eq 6.49	$\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1) =$ **0.22**
Design buckling resistance - eq 6.47	$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **542.6** kN
	$N_{Ed} / N_{b,z,Rd} =$ **0.126**
	PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m4\_s1\_seg1\_R} =$ **5700** mm
Distance from shear centre to centroid in y axis	$y_0 =$ **0.0** mm
Distance from shear centre to centroid in z axis	$z_0 =$ **0.0** mm
Radius of gyration	$i_0 = \sqrt{(i_y^2 + i_z^2)} =$ **169.3** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2 =$ **1557.3** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2 =$ **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF} =$ **1557.3** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **1557.3** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})} =$ **1.258**

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

Buckling curve - Table 6.2	b
Imperfection factor - Table 6.1	$\alpha_T =$ **0.34**
Buckling reduction determination factor	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) =$ **1.471**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1) =$ **0.448**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **1103.4** kN
	$N_{Ed} / N_{b,T,Rd} =$ **0.062**
	PASS - Design buckling resistance exceeds design compression

Check design at start of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f =$ **380.8** mm $\quad \eta =$ **1.000**
	$h_w / t_w = 49.5 = 60.8 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed} =$ **42** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **3431** mm^2
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **703.3** kN
	$V_{y,Ed} / V_{c,y,Rd} =$ **0.06**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed}$ = **123.4** kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **377.7** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.327**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.65**

$C_1 = 1 / k_c^2$ = **2.367**

Poissons ratio — ν = **0.3**

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

Unrestrained effective length — $L = 1.0 \times L_{m4\_s1\_seg1\_B}$ = **5700** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **403.8** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.967**

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 — c

Imperfection factor - Table 6.3 — α_{LT} = **0.49**

Correction factor for rolled sections — β = **0.75**

LTB reduction determination factor — $\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.990**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.659**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.835**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.790**

Design buckling resistance moment - eq 6.55 — $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **298.2** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.414**

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 — $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **520.5** kN

$N_{Ed} / N_{y,lim}$ = **0.131**

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 — ψ_y = 115.77 kNm / -123.405 kNm = **-0.938**

α_y = -3.818 kNm / -123.405 kNm = **0.031**

$C_{my} = \max(0.6 + 0.4 \times \psi_y, 0.4)$ = **0.400**

ψ_{LT} = 115.77 kNm / -123.405 kNm = **-0.938**

α_{LT} = -3.818 kNm / -123.405 kNm = **0.031**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance — $M_{y,Rk} = W_{pl,y} \times f_y$ = **377.7** kNm

Characteristic moment resistance — $M_{z,Rk} = W_{pl,z} \times f_y$ = **63.4** kNm

Characteristic resistance to normal force — $N_{Rk} = A \times f_y$ = **2465.3** kN

Interaction factors — $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.403**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.916**

Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.229**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.58**
	PASS - Combined bending and compression checks are satisfied

Check design at end of span

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f$ = **380.8** mm η = **1.000**
	$h_w / t_w = 49.5 = 60.8 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
	Shear buckling resistance can be ignored
Design shear force	$V_{y,Ed}$ = **42** kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **3431** mm²
Design shear resistance - cl 6.2.6(2)	$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **703.3** kN
	$V_{y,Ed} / V_{c,y,Rd}$ = **0.06**
	PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **115.8** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **377.7** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.307**
	PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.65**
	$C_1 = 1 / k_c^2$ = **2.367**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²
Unrestrained effective length	$L = 1.0 \times L_{m4\_s1\_seg1\_T}$ = **5700** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **403.8** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})}$ = **0.967**
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	c
Imperfection factor - Table 6.3	α_{LT} = **0.49**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.990**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.659**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.835**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.790**
Design buckling resistance moment - eq 6.55	$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1}$ = **298.2** kNm
	$M_{y,Ed} / M_{b,y,Rd}$ = **0.388**
	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	ψ_y = 115.77 kNm / -123.405 kNm = **-0.938**
	α_y = -3.818 kNm / -123.405 kNm = **0.031**

C_{my} = max(0.6 + 0.4 × ψ_y, 0.4) = **0.400**

ψ_{LT} = 115.77 kNm / -123.405 kNm = **-0.938**

α_{LT} = -3.818 kNm / -123.405 kNm = **0.031**

C_{mLT} = max(0.6 + 0.4 × ψ_{LT}, 0.4) = **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl,y} \times f_y$ = **377.7** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl,z} \times f_y$ = **63.4** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **2465.3** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.403**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.911**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.219**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.557**

PASS - Combined bending and compression checks are satisfied

Consider Combination 4 - 1.0G + 1.0Q + 0.5S + 0.5W (Service)

Check design 141 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection	δ_y = **10.9** mm
Allowable deflection	$\delta_{y,Allowable} = L_{m4\_s1} / 180$ = **31.7** mm
	$\delta_y / \delta_{y,Allowable}$ = **0.344**

PASS - Allowable deflection exceeds design deflection