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Job No: 11029DOL24

Agricultural Building No. 2
End Farm
Marston
Wiltshire

Report on the Structural Aspects of the feasibility of conversion of an agricultural building

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Agricultural Building No.2, End Farm, Marston, Wiltshire

Report on the Structural Aspects of the feasibility of conversion of an agricultural building at End Farm, Marston, Wiltshire

Purpose of the Report

Structural Solutions was commissioned to carry out an investigation into the feasibility of converting an agricultural building at End Farm, Marston into two residential properties. This report considers the current condition of the structure of the building and the likely work required as part of the conversion.

Introduction

The building is rectangular in plan approximately 13.5m x 13.4m divided into 3 'bays' of approximately 4.5m width each. The external elevations are clad with profiled metal sheets to three elevations and also to the south west elevation above a 'large' door opening but corrugated cement based sheets from ground to eaves level. Internally, the walls are typically concrete blockwork inside the external sheet cladding either 1650mm high or 750mm above ground level. The roof is clad with corrugated cement based sheet spanning between timber purlins which in turn span between the main structural steel frames.

Investigation

Our investigation is based on a visual walk over carried out on 10th March 2026.

The existing building is currently an agricultural structure used as general farm storage and stables, associated with the agricultural holding comprised of four structural steel portal frames forming three bays of approximately 4.5 metres in width. There is a frame in each of the end gable elevations and two internal. The frames are comprised of I-Section steel columns and principal rafters with bolted haunches at eaves and ridge to improve the bending capacity at the connections. The roof covering is a corrugated cement based sheet supported by timber purlins spanning between the structural portal frames. There is a small concrete block store to one corner of the building. Bracing within the plane of the roof working in combination with concrete blockwork walls between post provide longitudinal stability. The external elevations on three sides are clad with vertical metal sheets supported by horizontal timber cladding rails. Although the sheets extend down to just above ground level, internal concrete block walls varying in height between 1650mm and 750mm above internal floor level provide a level of protection. There is a 'large' open doorway to the south west elevation which has similar metal cladding above but corrugated cement base sheet either side.

We understand that the proposed cladding to the elevations will be similar to the existing with vertical profiled metal sheets above a brickwork plinth. The internal concrete blockwork will be retained. This will have the benefit of providing insulation and weatherproofing of the building with no impact on the structure.

The building has a ground bearing concrete floor which is generally in a good condition. Foundations are likely to be concrete pads to support the main steel portal frame columns and trench fill footings to support the perimeter concrete block walls. These were not visible but there were no outward signs of movement of the frames or the cladding that would signify ground movement or subsidence problems. The existing structure appears to be in good condition.

Discussion

We understand from the Client that the proposed conversion work of the building will maintain the basic plan of the building including retention of the existing structures; roof frames, purlins and concrete block walls.

The structure can be incorporated into a conversion of the building without any structural alterations. The existing wall cladding is to be replaced with new but following the same theme as the existing with breather membranes and insulation measures added externally. The south west elevation where there is a 'large' door opening will be infilled with horizontal rails spanning between existing columns and clad to match the proposed, but this work can be done without compromising the integrity of the existing structure, and without adding structure due to utilising the existing columns to take the loads.

The proposed conversion as shown on Dolman's drawings includes the construction of a first floor together with new internal partitions at ground floor and introduction of windows and doors into the existing external elevations. The existing concrete floor slab will be retained and new insulation and floor finishes will be introduced to support new internal partitions. New first floor construction and ceilings will be introduced without alteration or modification of the existing structure.

Conclusion

This report is not intended to be a full and detailed structural survey. Its purpose is to establish in structural terms whether the conversion of the agricultural building into usable residential properties is feasible without modification of the main structure. In brief, the existing roof structure, walls and foundations of the building can be incorporated into a conversion for residential use without modification. Similarly, the existing roof and wall cladding is capable of retention but that to improve weatherproofing and thermal performance will be changed but keeping with the same visual theme.

Wall linings will be provided to achieve appropriate residential standards secured to the inside face of the existing external walls. Insulation will be provided externally with composite cladding not exceeding 200mm thickness. The existing concrete ground floor slab will support new internal

partitions and first floor without compromising the integrity or having to modify the existing structure.

Outline Schedule of Works

The following is an outline Schedule of Works identifying anticipated elements of construction that will be retained and elements that will need to be constructed as new.

Elements for Retention

- Foundation of structural frame and concrete block walls (100%)
- Main concrete block external walls (100%)
- Main structural steel portal frames and purlins (100%)
- Concrete floor slab (100%)

Proposed Works/Modification

- Retain steel frame, foundations, floor slab and block walls for conversion.
- Infill south west elevation door opening using cladding rails spanning between existing columns and clad as proposed.
- Introduce external brickwork plinth below replacement wall cladding.
- Replace existing roof cladding with insulated profile metal sheeting.
- Introduce rooflights within roof cladding
- Linings to the inside face of the existing/retained external walls to give plasterboard finishes.
- Windows and doors openings to be introduced in accordance with the Architect's elevations.
- New floor insulation and screed finish above the existing ground bearing concrete floor structure.
- Timber frame partitions to support new first floor structure.
- New ceiling above first floor.

<u>Repairs/ Extent of Remedial Works</u>

<u>Walls</u>

- Inspect all blockwork. Allow for localised repairs.

<u>Roof</u>

- Internally, inspect and thoroughly clean existing structural steel portal frames and timber purlins and treat with suitable preservative. (100% of roof structure to be treated and painted).

<u>New Internal Works</u>

- New non-load bearing timber stud partitions to be constructed using 50x100 C16 grade softwood timber studs @ 600mm c/c's to form ground and first floor rooms. Partitions

and linings to be constructed off the existing ground bearing concrete slab to ensure no timber below finished floor level.

- New load bearing timber stud partitions to be constructed using 50x100 C16 grade softwood timber studs @ 600mm c/c's to support first floor structure. Partitions to be constructed off of the existing ground bearing concrete slab to ensure no timber below finished floor level.

- Services, e.g. electricity, plumbing, drainage as well as new architectural finishes to be coordinated as part of the overall conversion. No anticipated structural affect.

- All walls and ceilings to be insulated to the Architect's specification.

Appendix A – Site Photos

External south east elevation

External south west elevation

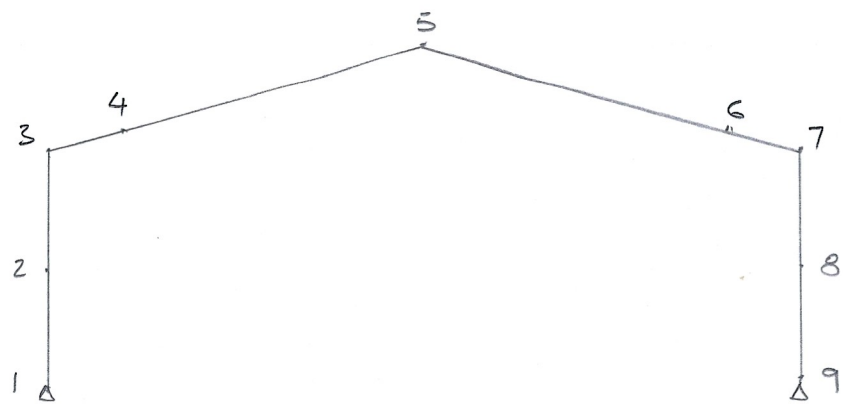
Internal view showing structural steel portal frame, roof & wall cladding

View of Haunch Connection between portal frame post & principal rafter

View of Haunch Connection at Ridge

Appendix B – Structural Calculations

Project	Sheet no./rev.
Barn N° 2, End Farm, Marston, Wiltshire	DOL24 9
Main Frame Analysis	By/Date MM Apr '26



Nodes	x	y
1	0	0
2	0	2.5
3	0	3.6
4	0.80	3.814
5	6.650	5.382
6	12.50	3.814
7	13.30	3.6
8	13.30	2.5
9	13.30	0

Spacing of frames 4500

Members

Post/Column 203x133x25 UB

Rafter 203x133x25 UB

+ Haunch @ Eaves & Ridge

Loading.

Dead 0.15 kN/m² × 4.5 = 0.68 kN/m Roof Rafter

Snow 0.17 kN/m² × 4.5 = 0.77 kN/m

Wind

Windward wall 0.57 kN/m² × 4.5m = 2.57 kN/m

Leeward wall 0.01 kN/m² × 4.5m = 0.05 kN/m

Windward Rafter 0.31 kN/m² × 4.5m = 1.40 kN/m

Leeward Rafter −0.11 kN/m² × 4.5m = −0.50 kN/m

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SNOW LOADING (EN1991)

SNOW LOADING

In accordance with EN1991-1-3:2003+A1:2015 incorporating corrigenda dated December 2004 and March 2009 and the UK national annex NA+A1:2015 to BS EN 1991-1-3:2003+A1:2015 incorporating Corrigendum No.1

Tedds calculation version 1.0.14

Characteristic ground snow load

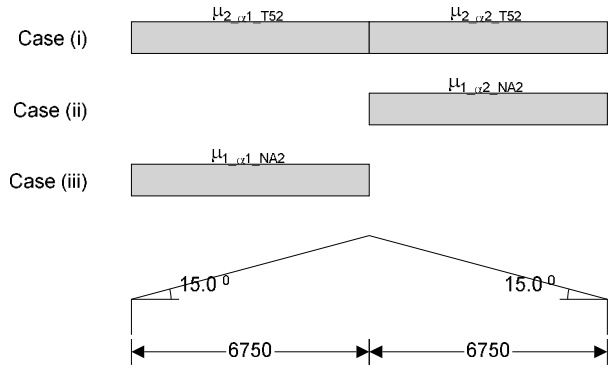
Location	Marston
Site altitude above sea level (user modified value)	A = **55** m
Zone number (user modified value)	Z = **1.0**
Density of snow	γ = **2.00** kN/m^3
Characteristic ground snow load	s_k = ((0.15 + (0.1 × Z + 0.05)) + ((A - 100m) / 525m)) × 1kN/m^2 = **0.21** kN/m^2
Exposure coefficient (Normal)	C_e = **1.0**
Thermal coefficient	C_t = **1.0**
Snow fence	Not present

Building details

Roof type	Duopitch
Width of roof (left on elevation)	b_1 = **6.75** m
Width of roof (right on elevation)	b_2 = **6.75** m
Slope of roof (left on elevation)	α_1 = **15.00** deg
Slope of roof (right on elevation)	α_2 = **15.00** deg

Shape coefficients

Shape coefficient roof (Table 5.2)	$\mu_{2_\alpha1_T52}$ = **0.80**
Shape coefficient roof (Table 5.2)	$\mu_{2_\alpha2_T52}$ = **0.80**
Shape coefficient roof (Table UK NA.2)	$\mu_{1_\alpha1_NA2}$ = **0.80**
Shape coefficient roof (Table UK NA.2)	$\mu_{1_\alpha2_NA2}$ = **0.80**



Elevation on gable roof

Shape coef	Coef	Loading (kN/m^2)
$\mu_{2_\alpha1_T52}$	0.800	0.17
$\mu_{2_\alpha2_T52}$	0.800	0.17
$\mu_{1_\alpha1_NA2}$	0.800	0.17
$\mu_{1_\alpha2_NA2}$	0.800	0.17

Loadcase 1 Table 5.2

Loading to roof 1 (LHS) $s_{1_1} = \mu_{2_\alpha1_T52} \times C_e \times C_t \times s_k =$ **0.17** kN/m^2

Loading to roof 2 (RHS) $s_{2_1} = \mu_{2_\alpha2_T52} \times C_e \times C_t \times s_k =$ **0.17** kN/m^2

Loadcase 2 UK NA.2

Loading to roof 1 (LHS) $s_{1_2} = 0 \times C_e \times C_t \times s_k =$ **0.00** kN/m^2

Loading to roof 2 (RHS) $s_{2_2} = \mu_{1_\alpha2_NA2} \times C_e \times C_t \times s_k =$ **0.17** kN/m^2

Loadcase 3 UK NA.2

Loading to roof 1 (LHS) $s_{1_3} = \mu_{1_\alpha1_NA2} \times C_e \times C_t \times s_k =$ **0.17** kN/m^2

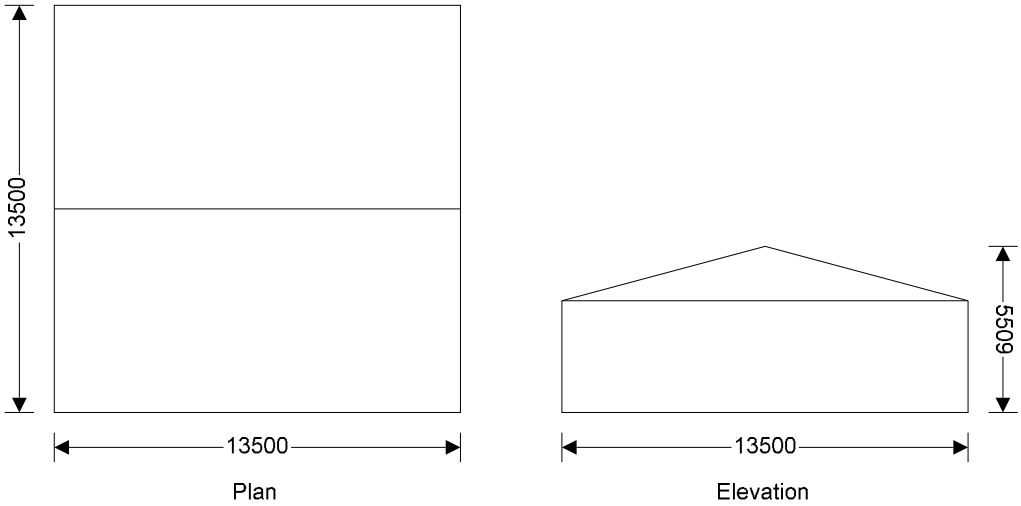
Loading to roof 2 (RHS) $s_{2_3} = 0 \times C_e \times C_t \times s_k =$ **0.00** kN/m^2

WIND LOADING (EN1991)

WIND LOADING

In accordance with EN1991-1-4:2005+A1:2010 and the UK national annex

Tedds calculation version 3.0.29



Building data

Type of roof	Duopitch
Length of building	L = **13500** mm
Width of building	W = **13500** mm
Height to eaves	H = **3700** mm
Pitch of roof	α_0 = **15.0** deg
Total height	h = **5509** mm

Basic values

Location	Marston
Wind speed velocity (FigureNA.1)	$v_{b,map}$ = **21.6** m/s
Distance to shore	L_{shore} = **70.00** km
Altitude above sea level	A_{alt} = **55.0**m
Altitude factor	$c_{alt} = A_{alt}/1m \times 0.001 + 1$ = **1.055**
Fundamental basic wind velocity	$v_{b,0} = v_{b,map} \times c_{alt}$ = **22.8** m/s
Direction factor	c_{dir} = **1.00**
Season factor	c_{season} = **1.00**
Shape parameter K	K = **0.2**
Exponent n	n = **0.5**
Air density	ρ = **1.226** kg/m^3
Probability factor	$c_{prob} = [(1 - K \times \ln(-\ln(1-p)))/(1 - K \times \ln(-\ln(0.98)))]^n$ = **1.00**
Basic wind velocity (Exp. 4.1)	$v_b = c_{dir} \times c_{season} \times v_{b,0} \times c_{prob}$ = **22.8** m/s
Reference mean velocity pressure	$q_b = 0.5 \times \rho \times v_b^2$ = **0.318** kN/m^2

Orography

Orography factor not significant c_o = 1.0

Terrain category Country

Displacement height (sheltering effect excluded) h_{dis} = 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.7.2.2)

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.7.2.2)

Peak velocity pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought) z = **3700**mm

Displacement height (sheltering effects excluded) h_{dis} = **0** mm

Exposure factor (Figure NA.7) c_e = **1.75**

Peak velocity pressure $q_p = c_e \times q_b$ = **0.56** kN/m^2

Structural factor

Structural damping δ_s = **0.050**

Height of element h_{part} = **3700** mm

Size factor (Table NA.3) c_s = **0.918**

Dynamic factor (Figure NA.9) c_d = **1.029**

Structural factor $c_{sCd} = c_s \times c_d$ = **0.945**

Peak velocity pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought) z = **5509**mm

Displacement height (sheltering effects excluded) h_{dis} = **0** mm

Exposure factor (Figure NA.7) c_e = **1.98**

Peak velocity pressure $q_p = c_e \times q_b$ = **0.63** kN/m^2

Structural factor

Structural damping δ_s = **0.050**

Height of element h_{part} = **5509** mm

Size factor (Table NA.3) c_s = **0.913**

Dynamic factor (Figure NA.9) c_d = **1.029**

Structural factor $c_{sCd} = c_s \times c_d$ = **0.939**

Structural factor - roof 0 deg

Structural damping δ_s = **0.050**

Height of element h_{part} = **5509** mm

Size factor (Table NA.3) c_s = **0.913**

Dynamic factor (Figure NA.9) c_d = **1.029**

Structural factor $c_{sCd} = c_s \times c_d$ = **0.939**

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) $q_{p,i}$ = **0.63** kN/m^2

Pressures and forces

Net pressure $p = c_{sCd} \times q_p \times c_{pe} - q_{p,i} \times c_{pi}$

Net force $F_w = p_w \times A_{ref}$

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

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (-ve)	-1.10	0.63	-0.78	6.28	-4.89
G (-ve)	-0.80	0.63	-0.60	9.11	-5.47
H (-ve)	-0.40	0.63	-0.36	78.94	-28.66
I (-ve)	-0.50	0.63	-0.42	78.94	-33.34
J (-ve)	-1.30	0.63	-0.90	15.40	-13.80

Total vertical net force $F_{w,v}$ = **-83.21** kN

Total horizontal net force $F_{w,h}$ = **2.10** kN

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

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.63	-0.84	8.80	-7.40
B	-0.80	0.63	-0.60	43.34	-26.13
C	-0.50	0.63	-0.42	10.01	-4.25
D	0.72	0.56	0.25	49.95	12.67
E	-0.34	0.56	-0.31	49.95	-15.30

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-15.3** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **12.7** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/W is 0.408

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **25.6** kN

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

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (+ve)	0.20	0.63	0.31	6.28	1.93
G (+ve)	0.20	0.63	0.31	9.11	2.80
H (+ve)	0.20	0.63	0.31	78.94	24.29
I (+ve)	-0.50	0.63	-0.11	78.94	-8.45
J (+ve)	-1.30	0.63	-0.58	15.40	-8.94

Total vertical net force $F_{w,v}$ = **11.24** kN

Total horizontal net force $F_{w,h}$ = **12.01** kN

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

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.63	-0.53	8.80	-4.63
B	-0.80	0.63	-0.29	43.34	-12.46
C	-0.50	0.63	-0.11	10.01	-1.09
D	0.72	0.56	0.57	49.95	28.42
E	-0.34	0.56	0.01	49.95	0.45

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **0.4** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **28.4** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/W is 0.408

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **34.0** kN

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

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (-ve)	-1.60	0.63	-1.07	6.28	-6.75
G (-ve)	-1.50	0.63	-1.01	9.11	-9.25
H (-ve)	-0.60	0.63	-0.48	61.59	-29.66
I (-ve)	-0.40	0.63	-0.36	111.69	-40.55

Total vertical net force $F_{w,v}$ = **-83.26** 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}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.56	-0.75	8.15	-6.15
B	-0.80	0.56	-0.54	32.61	-17.77
C	-0.50	0.56	-0.39	9.19	-3.56
D	0.72	0.63	0.30	62.16	18.71
E	-0.34	0.63	-0.33	62.16	-20.44

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-20.4** kN

Net windward force for overall section $F_w = F_{w,wD}$ = **18.7** kN

Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/L is 0.408

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **33.3** kN

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

Zone	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
F (+ve)	0.20	0.63	0.31	6.28	1.93
G (+ve)	0.20	0.63	0.31	9.11	2.80
H (+ve)	0.20	0.63	0.31	61.59	18.95
I (+ve)	0.20	0.63	0.31	111.69	34.36

Total vertical net force $F_{w,v}$ = **56.07** 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}	Peak velocity pressure q_p, (kN/m^2)	Net pressure p (kN/m^2)	Area A_{ref} (m^2)	Net force F_w (kN)
A	-1.20	0.56	-0.44	8.15	-3.58
B	-0.80	0.56	-0.23	32.61	-7.49
C	-0.50	0.56	-0.07	9.19	-0.67
D	0.72	0.63	0.62	62.16	38.31
E	-0.34	0.63	-0.01	62.16	-0.84

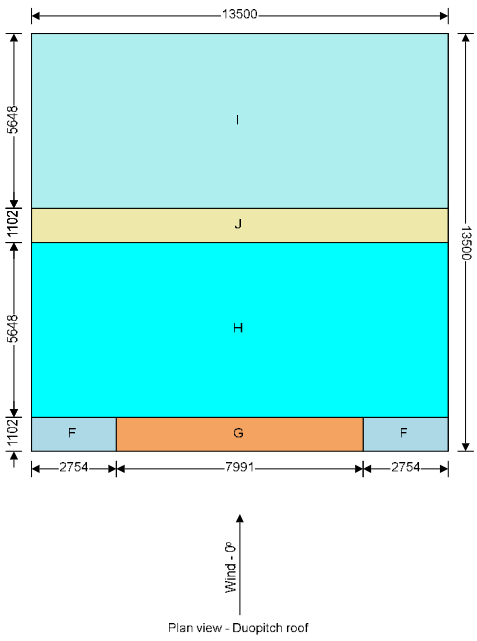
Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wE}$ = **-0.8** kN

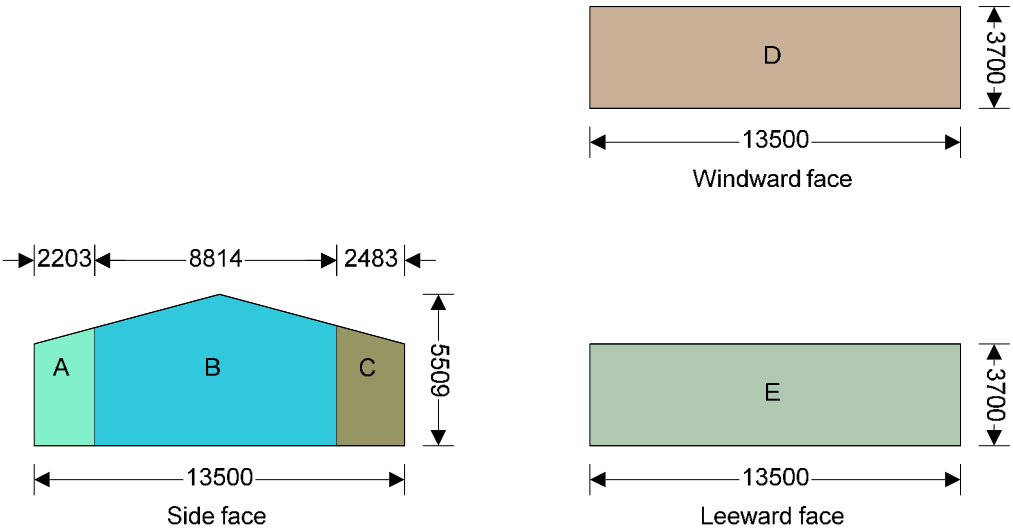
Net windward force for overall section $F_w = F_{w,wD}$ = **38.3** kN

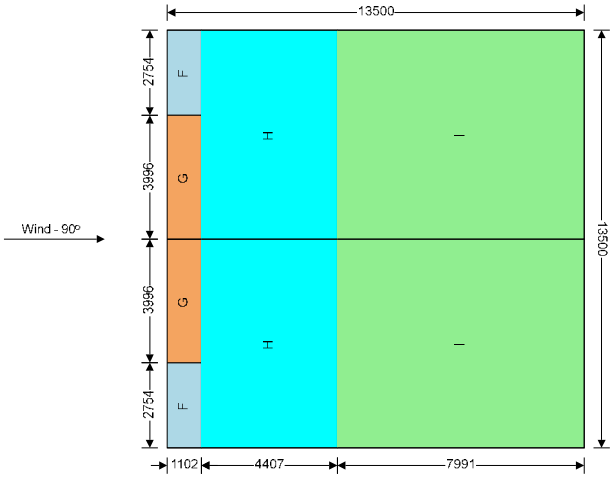
Lack of correlation (cl.7.2.2(3) – Note) f_{corr} = **0.85** as h/L is 0.408

Overall loading overall section $F_{w,D} = f_{corr} \times (F_w - F_l + F_{w,h})$ = **33.3** kN



Project	Sheet no./rev.
Agricultural Building No. 2, End Farm, Marston, Wiltshire	DOL24 / 17
Building Wind Load	By/Date MM / Apr-26





Plan view - Duopitch roof

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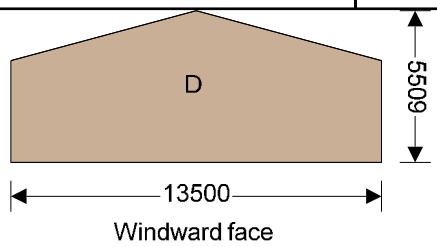
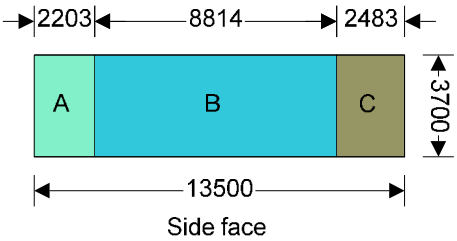
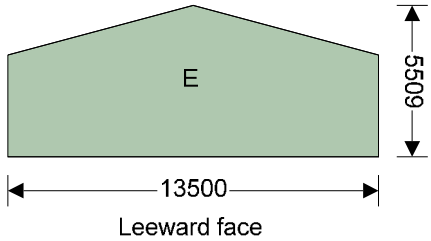
Agricultural Building No. 2, End Farm, Marston, Wiltshire	DOL24 / 18
Building Wind Load	By/Date MM / Apr-26







STRUCTURAL SOLUTIONS

STEEL 2D ANALYSIS & DESIGN (EN1993)

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

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

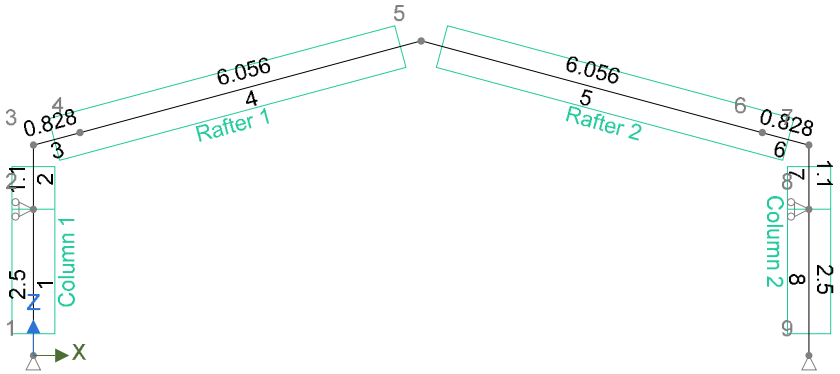
Tedds calculation version 4.5.03

ANALYSIS

Tedds calculation version 1.0.38

Geometry

Geometry (m) - Steel (EC3) - UB 203x133x25



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)
UB 203x133x25	32	2340.2	307.6	11.6	18.7

Project	Sheet no./rev.
Agricultural Barn No. 2, End Farm, Marston, Wiltshire	DOL24 / 20
Existing Portal Frame Analysis check	By/Date MM / Apr-26

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	Free		0	0	0	0
2	0	2.5	Fixed	Free	Free		0	0	0	0
3	0	3.6	Free	Free	Free		0	0	0	0
4	0.8	3.814	Free	Free	Free		0	0	0	0
5	6.65	5.382	Free	Free	Free		0	0	0	0
6	12.5	3.814	Free	Free	Free		0	0	0	0
7	13.3	3.6	Free	Free	Free		0	0	0	0
8	13.3	2.5	Fixed	Free	Free		0	0	0	0
9	13.3	0	Fixed	Fixed	Free		0	0	0	0

Elements

Element	Length (m)	Nodes		Section	Material	Releases			Rotated
		Start	End			Start moment	End moment	Axial	
1	2.5	1	2	UB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	
2	1.1	2	3	UB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	
3	0.828	3	4	UB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	
4	6.056	4	5	UB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	
5	6.056	5	6	UB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	
6	0.828	6	7	UB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	
7	1.1	7	8	UB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	
8	2.5	8	9	UB 203x133x25	Steel (EC3)	Fixed	Fixed	Fixed	

Members

Name	Elements	
	Start	End
Column 1	1	2
Rafter 1	3	4

STRUCTURAL SOLUTIONS

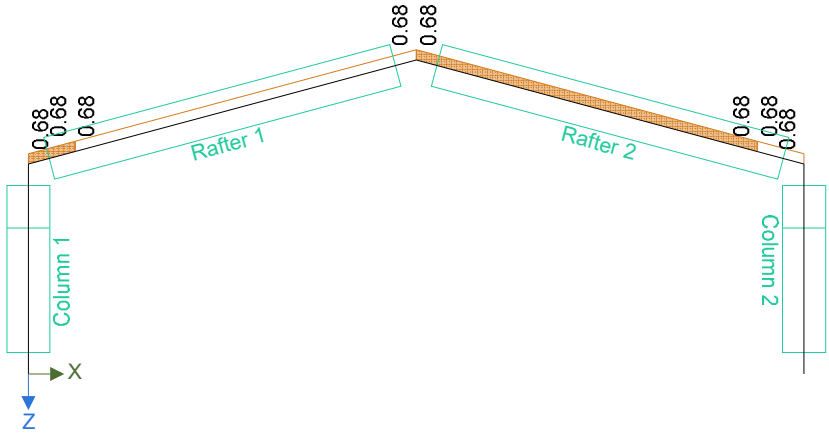
Project	Sheet no./rev.
Agricultural Barn No. 2, End Farm, Marston, Wiltshire	DOL24 / 21
Existing Portal Frame Analysis check	By/Date MM / Apr-26

Name	Elements	
	Start	End
Rafter 2	5	6
Column 2	7	8

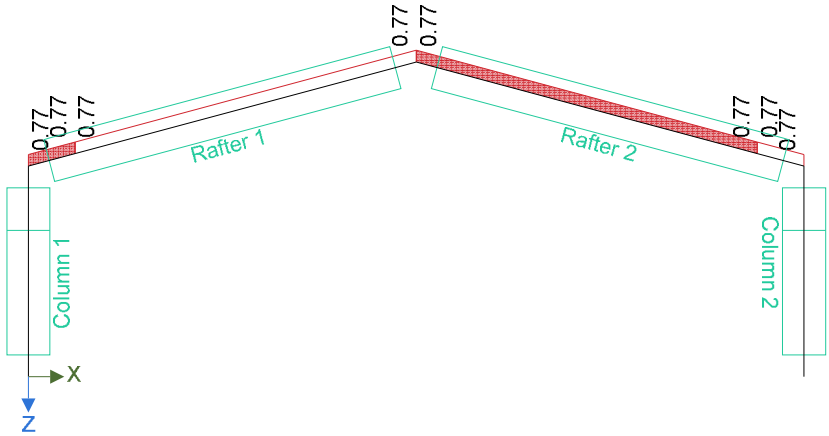
Loading

Self weight included

Permanent - Loading (kN/m)

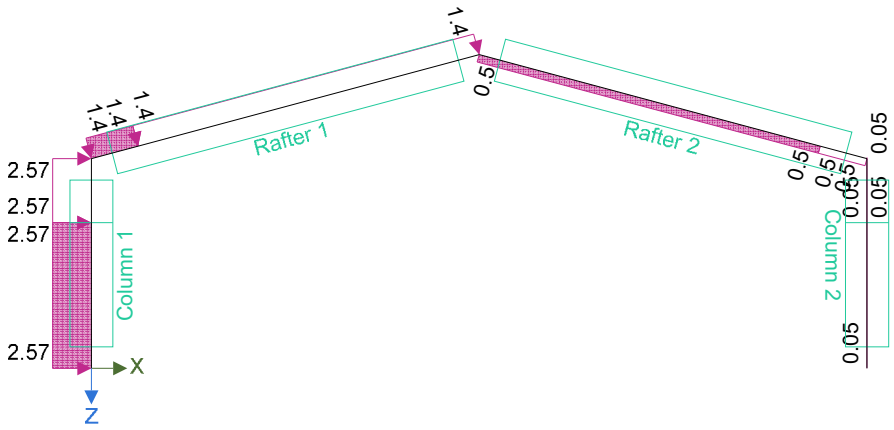


Snow - Loading (kN/m)



STRUCTURAL SOLUTIONS

Wind - Loading (kN/m)



Load combination factors

Load combination	Self Weight	Permanent	Snow	Wind
1.35G + 1.5ψ₀Q + 1.5S (Strength)	**1.35**	**1.35**	**1.50**	
1.35G + 1.5ψ₀Q + 1.5S + 1.5ψ₀W (Strength)	**1.35**	**1.35**	**1.50**	**0.75**
1.0G + 1.0ψ₀Q + 1.0S + 0.5W (Service)	**1.00**	**1.00**	**1.00**	**0.50**
1.35G + 1.5ψ₀Q + 1.5ψ₀S + 1.5W (Strength)	**1.35**	**1.35**	**0.75**	**1.50**
1.0G + 1.0ψ₀Q + 0.5S + 1.0W (Service)	**1.00**	**1.00**	**0.50**	**1.00**
1.0G + 1.5W (Strength)	**1.00**	**1.00**		**1.50**
1.0G + 1.0W (Service)	**1.00**	**1.00**		**1.00**

Member Loads

Member	Load case	Load Type	Orientation	Description
Rafter 1	Permanent	UDL	GlobalZ	0.68 kN/m
Rafter 2	Permanent	UDL	GlobalZ	0.68 kN/m
Rafter 1	Snow	UDL	GlobalZ	0.77 kN/m
Rafter 2	Snow	UDL	GlobalZ	0.77 kN/m
Column 1	Wind	UDL	GlobalX	2.57 kN/m
Rafter 1	Wind	UDL	LocalZ	1.4 kN/m
Rafter 2	Wind	UDL	LocalZ	-0.5 kN/m
Column 2	Wind	UDL	LocalX	0.05 kN/m

Results

Reactions

Load case: Self Weight

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-0.4	2.6	0
2	2.7	0	0
8	-2.7	0	0
9	0.4	2.6	0

Load case: Permanent

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-1	4.7	0
2	7.6	0	0
8	-7.6	0	0
9	1	4.7	0

Load case: Snow

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-1.1	5.3	0
2	8.6	0	0
8	-8.6	0	0
9	1.1	5.3	0

Load case: Wind

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-1.5	6.3	0
2	-5.5	0	0
8	-8	0	0
9	2.3	-0.1	0

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

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-3.6	17.8	0
2	26.7	0	0
8	-26.7	0	0
9	3.6	17.8	0

Load combination: 1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-4.7	22.5	0
2	22.7	0	0
8	-32.7	0	0
9	5.3	17.7	0

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

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-3.3	15.7	0
2	16.1	0	0

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
8	-22.9	0	0
9	3.7	12.5	0

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

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-5	23.2	0
2	12.1	0	0
8	-32.3	0	0
9	6.2	13.6	0

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

Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-3.4	16.2	0
2	9.1	0	0
8	-22.6	0	0
9	4.3	9.8	0

Load combination: 1.0G + 1.5W (Strength)

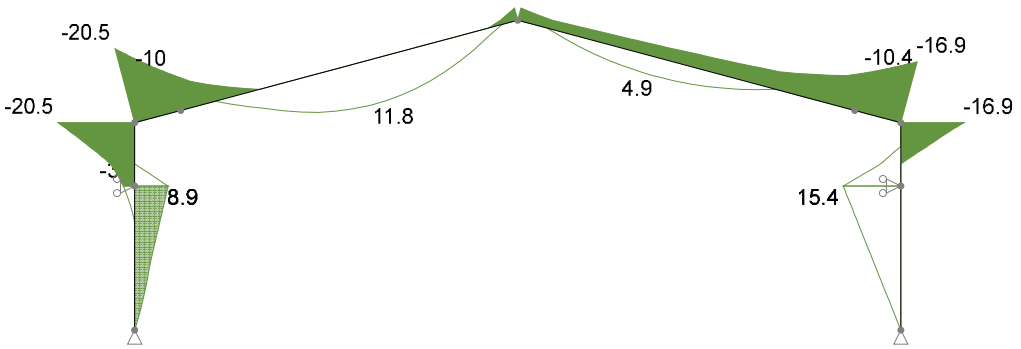
Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-3.6	16.7	0
2	2.1	0	0
8	-22.3	0	0
9	4.8	7.1	0

Load combination: 1.0G + 1.0W (Service)

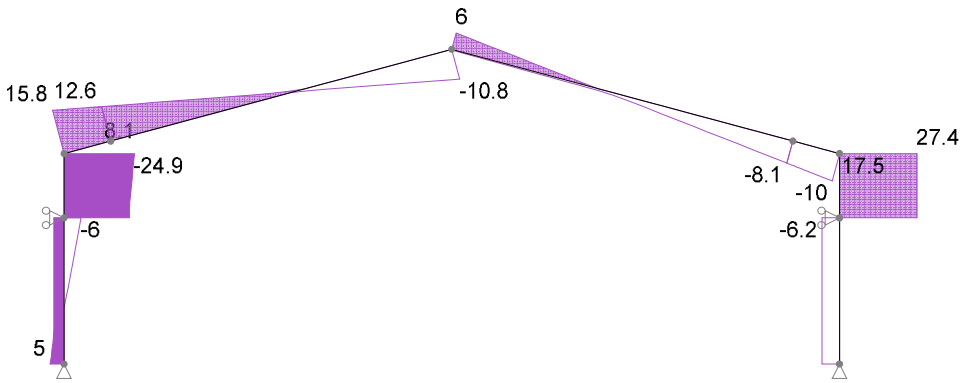
Node	Force		Moment
	Fx (kN)	Fz (kN)	My (kNm)
1	-2.9	13.5	0
2	4.8	0	0
8	-18.3	0	0
9	3.7	7.1	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



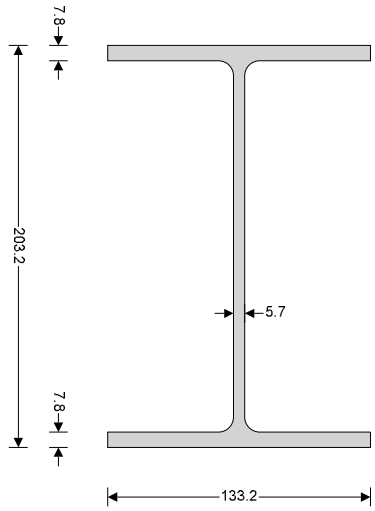
Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1**
Resistance of members to instability	γ_{M1} = **1**
Resistance of tensile members to fracture	γ_{M2} = **1.1**

Column 1 - Span 1 design

Section details

Section type	UB 203x133x25 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_f, t_w) =$ **7.8** 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 203x133x25 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 203.2 mm
Section breadth, b, 133.2 mm
Mass of section, Mass, 25.1 kg/m
Flange thickness, t_f, 7.8 mm
Web thickness, t_w, 5.7 mm
Root radius, r, 7.6 mm
Area of section, A, 3197 mm^2
Radius of gyration about y-axis, i_y, 85.559 mm
Radius of gyration about z-axis, i_z, 31.021 mm
Elastic section modulus about y-axis, $W_{el,y}$, 230332 mm^3
Elastic section modulus about z-axis, $W_{el,z}$, 46190 mm^3
Plastic section modulus about y-axis, $W_{pl,y}$, 257731 mm^3
Plastic section modulus about z-axis, $W_{pl,z}$, 70944 mm^3
Second moment of area about y-axis, I_y, 23401694 mm^4
Second moment of area about z-axis, I_z, 3076268 mm^4

Lateral restraint

Left flange has full lateral restraint

Right flange has lateral restraint at supports only

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

Classification of cross sections - Section 5.5

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

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.533**

$c / t_w = 30.2 = 32.7 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 7.2 = 7.8 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force	$N_{Ed} =$ **17.8** kN
Design resistance of section - eq 6.10	$N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **879.1** kN
	$N_{Ed} / N_{c,Rd} =$ **0.02**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length $L_{cr,y} = L_{m1_s1}$ = **2500** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **7760.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.337**

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 $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **0.571**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.969**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **851.7** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.021**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length $L_{cr,T} = L_{m1_s1_seg1_R}$ = **2500** 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 $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **91.0** mm

Elastic critical torsional buckling force $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **1757.3** kN

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

Elastic critical torsional-flexural buckling force $N_{cr,TF}$ = **1757.3** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **1757.3** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.707**

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 $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.836**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.78**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **685.3** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.026**

PASS - Design buckling resistance exceeds design compression

<u>Check design at end of span</u>

Check shear - Section 6.2.6

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

$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

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

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1282** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **203.5** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.018**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **8.9** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **70.9** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.126**

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 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **147.0** kN

$N_{Ed} / N_{y,lim}$ = **0.115**

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 = 0 kNm / 8.932 kNm = **0.000**

α_y = 4.466 kNm / 8.932 kNm = **0.500**

$C_{my} = \max(0.6 + 0.4 \times \psi_y)$ = **0.600**

ψ_{LT} = 0 kNm / 8.932 kNm = **0.000**

α_{LT} = 4.466 kNm / 8.932 kNm = **0.500**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT})$ = **0.600**

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

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y$ = **70.9** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y$ = **19.5** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y$ = **879.1** kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.602**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.992**

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.096**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.155**

PASS - Combined bending and compression checks are satisfied

<u>Column 1 - Span 2 design</u>

Section details

Section type UB 203x133x25 (British Steel Section Range 2022 (BS4-1))

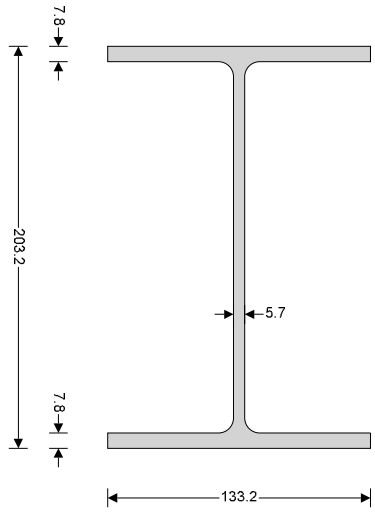
Steel grade - EN 10025-2:2004 S275

Nominal thickness of element $t_{nom} = \max(t_f, t_w)$ = **7.8** mm

Nominal yield strength f_y = **275** N/mm^2

Nominal ultimate tensile strength f_u = **410** N/mm^2

Modulus of elasticity $E = \mathbf{210000}$ N/mm^2



UB 203x133x25 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 203.2 mm
Section breadth, b, 133.2 mm
Mass of section, Mass, 25.1 kg/m
Flange thickness, t_f, 7.8 mm
Web thickness, t_w, 5.7 mm
Root radius, r, 7.6 mm
Area of section, A, 3197 mm^2
Radius of gyration about y-axis, i_y, 85.559 mm
Radius of gyration about z-axis, i_z, 31.021 mm
Elastic section modulus about y-axis, $W_{el,y}$, 230332 mm^3
Elastic section modulus about z-axis, $W_{el,z}$, 46190 mm^3
Plastic section modulus about y-axis, $W_{pl,y}$, 257731 mm^3
Plastic section modulus about z-axis, $W_{pl,z}$, 70944 mm^3
Second moment of area about y-axis, I_y, 23401694 mm^4
Second moment of area about z-axis, I_z, 3076268 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

Classification of cross sections - Section 5.5

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

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

Width of section $c = d = \mathbf{172.4}$ mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) = \mathbf{0.541}$

$c / t_w = 30.2 = 32.7 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 7.2 = 7.8 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force $N_{Ed} = \mathbf{22.4}$ kN

Design resistance of section - eq 6.10 $N_{c,Rd} = A \times f_y / \gamma_{M0} = \mathbf{879.1}$ kN

$N_{Ed} / N_{c,Rd} = \mathbf{0.025}$

PASS - Design compression resistance exceeds design compression

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

Critical buckling length $L_{cr,y} = L_{m1_s2} = \mathbf{1100}$ mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 = \mathbf{40084.9}$ kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} = \mathbf{0.148}$

Check y-y axis flexural buckling resistance - Section 6.3.1.1

Buckling curve - Table 6.2 a

Imperfection factor - Table 6.1 $\alpha_y = \mathbf{0.21}$

Buckling reduction determination factor $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) = \mathbf{0.506}$

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

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **879.1** kN

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

PASS - Design buckling resistance exceeds design compression

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

Critical buckling length $L_{cr,z} = L_{m1_s2_seg1} =$ **1100** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **5269.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **0.408**

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 $\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **0.619**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1) =$ **0.923**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **811.2** kN

$N_{Ed} / N_{b,z,Rd} =$ **0.028**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length $L_{cr,T} = L_{m1_s2_seg1_R} =$ **1100** 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 $i_0 = \sqrt{(i_y^2 + i_z^2)} =$ **91.0** mm

Elastic critical torsional buckling force $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2 =$ **6654.3** kN

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

Elastic critical torsional-flexural buckling force $N_{cr,TF} =$ **1757.3** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **1757.3** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})} =$ **0.707**

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 $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) =$ **0.836**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1) =$ **0.78**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **685.3** kN

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

PASS - Design buckling resistance exceeds design compression

<u>Check design at start of span</u>

Check shear - Section 6.2.6

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

$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **16.8** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1282** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **203.5** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.083**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **0.3** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **70.9** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.005**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

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

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

Poissons ratio ν = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m1_s2_seg1_T}$ = **1100** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **538.9** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.363**

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 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **147.0** kN

$N_{Ed} / N_{y,lim}$ = **0.152**

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 = 0.338 kNm / -20.509 kNm = **-0.016**

α_y = -9.502 kNm / -20.509 kNm = **0.463**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4)$ = **0.571**

ψ_{LT} = 0.338 kNm / -20.509 kNm = **-0.016**

α_{LT} = -9.502 kNm / -20.509 kNm = **0.463**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.571**

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

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y$ = **70.9** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y$ = **19.5** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y$ = **879.1** kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.570**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.996**

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.028**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.032**

PASS - Combined bending and compression checks are satisfied

<u>Check design at end of span</u>

Check shear - Section 6.2.6

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

$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed} =$ **21.1** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1282** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **203.5** kN

$V_{y,Ed} / V_{c,y,Rd} =$ **0.104**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed} =$ **20.5** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} =$ **70.9** kNm

$M_{y,Ed} / M_{c,y,Rd} =$ **0.289**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

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

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

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

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

Unrestrained effective length $L = 1.0 \times L_{m1_s2_seg1_B} =$ **1100** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **538.9** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} =$ **0.363**

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 $\psi_y = 0.338$ kNm / -20.509 kNm = **-0.016**

$\alpha_y = -9.502$ kNm / -20.509 kNm = **0.463**

$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4) =$ **0.571**

$\psi_{LT} = 0.338$ kNm / -20.509 kNm = **-0.016**

α_{LT} = -9.502 kNm / -20.509 kNm = **0.463**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.571**

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

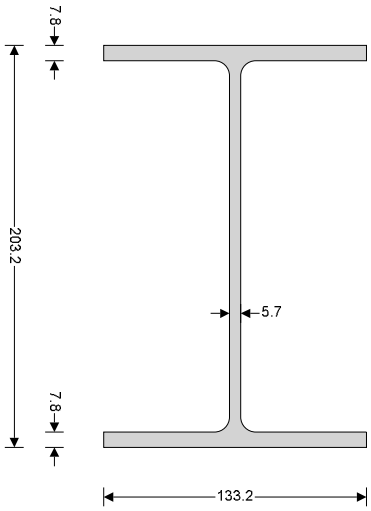
Characteristic moment resistance	$M_{y,Rk} = W_{pl.y} \times f_y$ = **70.9** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl.z} \times f_y$ = **19.5** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **879.1** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.570**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.997**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.19**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.315**

PASS - Combined bending and compression checks are satisfied

Rafter 1 design

Section details

Section type	UB 203x133x25 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **7.8** 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 203x133x25 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 203.2 mm
Section breadth, b, 133.2 mm
Mass of section, Mass, 25.1 kg/m
Flange thickness, t_f, 7.8 mm
Web thickness, t_w, 5.7 mm
Root radius, r, 7.6 mm
Area of section, A, 3197 mm^2
Radius of gyration about y-axis, i_y, 85.559 mm
Radius of gyration about z-axis, i_z, 31.021 mm
Elastic section modulus about y-axis, $W_{el.y}$, 230332 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 46190 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 257731 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 70944 mm^3
Second moment of area about y-axis, I_y, 23401694 mm^4
Second moment of area about z-axis, I_z, 3076268 mm^4

Lateral restraint

Both flanges have full lateral restraint

Consider Combination 2 - 1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (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

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.555**

$c / t_w = 30.2 = 32.7 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 7.2 = 7.8 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force N_{Ed} = **29.6** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = A \times f_y / \gamma_{M0}$ = **879.1** kN

$N_{Ed} / N_{c,Rd}$ = **0.034**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length $L_{cr,y} = L_{m2_s1}$ = **6885** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **1023.3** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.927**

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 $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **1.006**

Buckling reduction factor - eq 6.49 $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.716**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **629.5** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.047**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length $L_{cr,T} = L_{m2_s1_seg1_R}$ = **6885** 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 $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **91.0** mm

Elastic critical torsional buckling force $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **736.6** kN

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

Elastic critical torsional-flexural buckling force $N_{cr,TF}$ = **1757.3** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **736.6** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **1.092**

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 $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **1.248**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.54**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **474.5** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.062**

PASS - Design buckling resistance exceeds design compression

<u>Check design at start of span</u>

Check shear - Section 6.2.6

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

$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **14.1** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1282** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **203.5** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.069**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **20.5** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **70.9** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.29**

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 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **147.0** kN

$N_{Ed} / N_{y,lim}$ = **0.201**

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 = -3.388 kNm / -20.523 kNm = **0.165**

α_y = 8.036 kNm / -20.523 kNm = **-0.392**

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

ψ_{LT} = -3.388 kNm / -20.523 kNm = **0.165**

α_{LT} = 8.036 kNm / -20.523 kNm = **-0.392**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y$ = **70.9** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y$ = **19.5** kNm

Characteristic resistance to normal force	$N_{Rk} = A \times f_y =$ **879.1** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.414**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.990**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.167**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.323**

PASS - Combined bending and compression checks are satisfied

Check design 4180 mm along span

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed} =$ **9** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} =$ **70.9** kNm
	$M_{y,Ed} / M_{c,y,Rd} =$ **0.126**

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	$\psi_y =$ -3.388 kNm / -20.523 kNm = **0.165**
	$\alpha_y =$ 8.036 kNm / -20.523 kNm = **-0.392**
	$C_{my} = \max(0.2 + 0.8 \times \alpha_y, 0.4) =$ **0.400**
	$\psi_{LT} =$ -3.388 kNm / -20.523 kNm = **0.165**
	$\alpha_{LT} =$ 8.036 kNm / -20.523 kNm = **-0.392**
	$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4) =$ **0.400**

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

Characteristic moment resistance	$M_{y,Rk} = W_{pl.y} \times f_y =$ **70.9** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl.z} \times f_y =$ **19.5** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y =$ **879.1** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.412**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) =$ **0.991**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.095**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.158**

PASS - Combined bending and compression checks are satisfied

Rafter 2 design

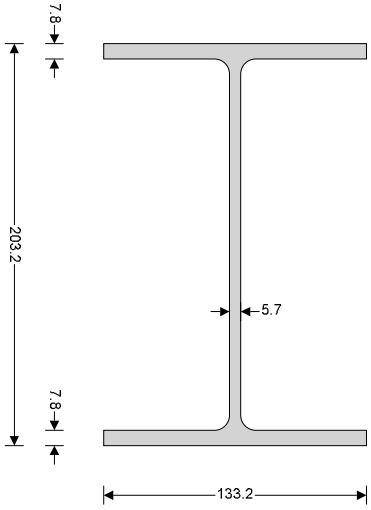
Section details

Section type	UB 203x133x25 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_f, t_w) =$ **7.8** 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 203x133x25 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 203.2 mm
Section breadth, b, 133.2 mm
Mass of section, Mass, 25.1 kg/m
Flange thickness, t_f, 7.8 mm
Web thickness, t_w, 5.7 mm
Root radius, r, 7.6 mm
Area of section, A, 3197 mm^2
Radius of gyration about y-axis, i_y, 85.559 mm
Radius of gyration about z-axis, i_z, 31.021 mm
Elastic section modulus about y-axis, $W_{el,y}$, 230332 mm^3
Elastic section modulus about z-axis, $W_{el,z}$, 46190 mm^3
Plastic section modulus about y-axis, $W_{pl,y}$, 257731 mm^3
Plastic section modulus about z-axis, $W_{pl,z}$, 70944 mm^3
Second moment of area about y-axis, I_y, 23401694 mm^4
Second moment of area about z-axis, I_z, 3076268 mm^4

Lateral restraint

Both flanges have full lateral restraint

<u>Consider Combination 2 - 1.35G + 1.5ψ_0Q + 1.5S + 1.5ψ_0W (Strength)</u>

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

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1)$ = **0.557**

$c / t_w = 30.2 = 32.7 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 7.2 = 7.8 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force N_{Ed} = **26.5** kN

Design resistance of section - eq 6.10 $N_{c,Rd} = A \times f_y / \gamma_{M0}$ = **879.1** 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 $L_{cr,y} = L_{m3_s1}$ = **6885** mm

Critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2$ = **1023.3** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **0.927**

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	$\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2)$ = **1.006**
Buckling reduction factor - eq 6.49	$\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1)$ = **0.716**
Design buckling resistance - eq 6.47	$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **629.5** kN
	$N_{Ed} / N_{b,y,Rd}$ = **0.042**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length	$L_{cr,T} = L_{m3_s1_seg1_R}$ = **6885** 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	$i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **91.0** mm
Elastic critical torsional buckling force	$N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **736.6** kN
Torsion factor	$\beta_T = 1 - (y_0 / i_0)^2$ = **1**
Elastic critical torsional-flexural buckling force	$N_{cr,TF}$ = **1757.3** kN
Elastic critical buckling force	$N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **736.6** kN
Slenderness ratio for torsional buckling - eq 6.52	$\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **1.092**

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	$\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **1.248**
Buckling reduction factor - eq 6.49	$\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.54**
Design buckling resistance - eq 6.47	$N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **474.5** kN
	$N_{Ed} / N_{b,T,Rd}$ = **0.056**

PASS - Design buckling resistance exceeds design compression

<u>Check design 2433 mm along span</u>

Check bending moment - Section 6.2.5

Design bending moment	$M_{y,Ed}$ = **2.4** kNm
Design bending resistance moment - eq 6.13	$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0}$ = **70.9** kNm
	$M_{y,Ed} / M_{c,y,Rd}$ = **0.034**

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 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **147.0** kN

$N_{Ed} / N_{y,lim}$ = **0.19**

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 = -3.388 kNm / -16.937 kNm = **0.200**

α_y = 1.387 kNm / -16.937 kNm = **-0.082**

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

ψ_{LT} = -3.388 kNm / -16.937 kNm = **0.200**

α_{LT} = 1.387 kNm / -16.937 kNm = **-0.082**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y$ = **70.9** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y$ = **19.5** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y$ = **879.1** kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.413**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.991**

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.058**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.068**

PASS - Combined bending and compression checks are satisfied

<u>Check design at end of span</u>

Check shear - Section 6.2.6

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

$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **8.7** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1282** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **203.5** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.043**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **16.9** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **70.9** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.239**

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 = -3.388 kNm / -16.937 kNm = **0.200**

α_y = 1.387 kNm / -16.937 kNm = **-0.082**

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

ψ_{LT} = -3.388 kNm / -16.937 kNm = **0.200**

α_{LT} = 1.387 kNm / -16.937 kNm = **-0.082**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{LT}, 0.4)$ = **0.400**

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

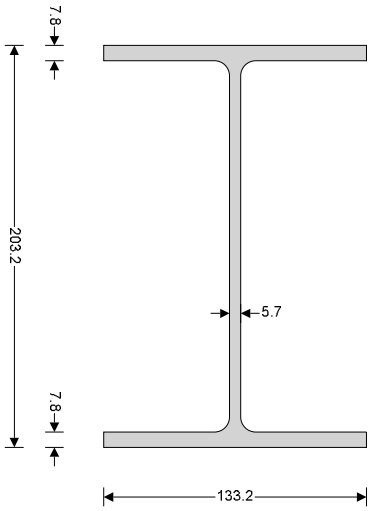
Characteristic moment resistance	$M_{y,Rk} = W_{pl.y} \times f_y$ = **70.9** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl.z} \times f_y$ = **19.5** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **879.1** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.414**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.990**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.148**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.274**

PASS - Combined bending and compression checks are satisfied

Column 2 - Span 1 design

Section details

Section type	UB 203x133x25 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **7.8** 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 203x133x25 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 203.2 mm
Section breadth, b, 133.2 mm
Mass of section, Mass, 25.1 kg/m
Flange thickness, t_f, 7.8 mm
Web thickness, t_w, 5.7 mm
Root radius, r, 7.6 mm
Area of section, A, 3197 mm^2
Radius of gyration about y-axis, i_y, 85.559 mm
Radius of gyration about z-axis, i_z, 31.021 mm
Elastic section modulus about y-axis, $W_{el.y}$, 230332 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 46190 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 257731 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 70944 mm^3
Second moment of area about y-axis, I_y, 23401694 mm^4
Second moment of area about z-axis, I_z, 3076268 mm^4

Lateral restraint

Left flange has full lateral restraint

Right flange has lateral restraint at supports only

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

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.531**

$c / t_w = 30.2 = 32.7 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 7.2 = 7.8 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force — $N_{Ed} =$ **16.6** kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **879.1** kN

$N_{Ed} / N_{c,Rd} =$ **0.019**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length — $L_{cr,y} = L_{m4_s1} =$ **1100** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **40084.9** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.148**

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 — $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.506**

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

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **879.1** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.019**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length — $L_{cr,T} = L_{m4_s1_seg1_R} =$ **1100** 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 — $i_0 = \sqrt{(i_y^2 + i_z^2)} =$ **91.0** mm

Elastic critical torsional buckling force — $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2 =$ **6654.3** kN

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

Elastic critical torsional-flexural buckling force — $N_{cr,TF} =$ **1757.3** kN

Elastic critical buckling force — $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **1757.3** kN

Slenderness ratio for torsional buckling - eq 6.52 — $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})} =$ **0.707**

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 — $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2) =$ **0.836**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1) =$ **0.78**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1} =$ **685.3** kN

$N_{Ed} / N_{b,T,Rd} =$ **0.024**

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**

$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed} =$ **23.2** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1282** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **203.5** kN

$V_{y,Ed} / V_{c,y,Rd} =$ **0.114**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed} =$ **16.6** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} =$ **70.9** kNm

$M_{y,Ed} / M_{c,y,Rd} =$ **0.234**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

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

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

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

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

Unrestrained effective length $L = 1.0 \times L_{m4_s1_seg1_B} =$ **1100** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} =$ **538.9** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **0.363**

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 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0}) =$ **147.0** kN

$N_{Ed} / N_{y,lim} =$ **0.113**

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

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 $\psi_y = 8.932$ kNm / -16.561 kNm = **-0.539**

$\alpha_y = -3.814$ kNm / -16.561 kNm = **0.230**

$C_{my} = \max(0.6 + 0.4 \times \psi_y) =$ **0.384**

$\psi_{LT} = 8.932$ kNm / -16.561 kNm = **-0.539**

$\alpha_{LT} = -3.814$ kNm / -16.561 kNm = **0.230**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}) =$ **0.384**

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

Characteristic moment resistance — $M_{y,Rk} = W_{pl.y} \times f_y =$ **70.9** kNm

Characteristic moment resistance — $M_{z,Rk} = W_{pl.z} \times f_y =$ **19.5** kNm

Characteristic resistance to normal force — $N_{Rk} = A \times f_y =$ **879.1** kN

Interaction factors — $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})) =$ **0.384**

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

Interaction formulae - eq 6.61 & eq 6.62 — $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.109**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) =$ **0.253**

PASS - Combined bending and compression checks are satisfied

<u>Check design at end of span</u>

Check shear - Section 6.2.6

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

$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed} =$ **23.2** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1282** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} =$ **203.5** kN

$V_{y,Ed} / V_{c,y,Rd} =$ **0.114**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed} =$ **8.9** kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} =$ **70.9** kNm

$M_{y,Ed} / M_{c,y,Rd} =$ **0.126**

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 — $\psi_y = 8.932$ kNm / -16.561 kNm = **-0.539**

$\alpha_y = -3.814$ kNm / -16.561 kNm = **0.230**

$C_{my} = \max(0.6 + 0.4 \times \psi_y) =$ **0.384**

$\psi_{LT} = 8.932$ kNm / -16.561 kNm = **-0.539**

$\alpha_{LT} = -3.814$ kNm / -16.561 kNm = **0.230**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT}) =$ **0.384**

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

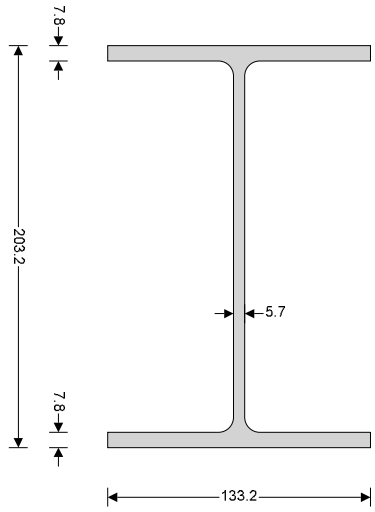
Characteristic moment resistance	$M_{y,Rk} = W_{pl.y} \times f_y$ = **70.9** kNm
Characteristic moment resistance	$M_{z,Rk} = W_{pl.z} \times f_y$ = **19.5** kNm
Characteristic resistance to normal force	$N_{Rk} = A \times f_y$ = **879.1** kN
Interaction factors	$k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.384**
	$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.994**
Interaction formulae - eq 6.61 & eq 6.62	$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.068**
	$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.146**

PASS - Combined bending and compression checks are satisfied

<u>Column 2 - Span 2 design</u>

Section details

Section type	UB 203x133x25 (British Steel Section Range 2022 (BS4-1))
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_f, t_w)$ = **7.8** 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 203x133x25 (British Steel Section Range 2022 (BS4-1))
Section depth, h, 203.2 mm
Section breadth, b, 133.2 mm
Mass of section, Mass, 25.1 kg/m
Flange thickness, t_f, 7.8 mm
Web thickness, t_w, 5.7 mm
Root radius, r, 7.6 mm
Area of section, A, 3197 mm^2
Radius of gyration about y-axis, i_y, 85.559 mm
Radius of gyration about z-axis, i_z, 31.021 mm
Elastic section modulus about y-axis, $W_{el.y}$, 230332 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 46190 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 257731 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 70944 mm^3
Second moment of area about y-axis, I_y, 23401694 mm^4
Second moment of area about z-axis, I_z, 3076268 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

<u>Consider Combination 4 - 1.35G + 1.5ψ_0Q + 1.5ψ_0S + 1.5W (Strength)</u>

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

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) =$ **0.525**

$c / t_w = 30.2 = 32.7 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 7.2 = 7.8 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check compression - Section 6.2.4

Design compression force — $N_{Ed} =$ **12.6** kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = A \times f_y / \gamma_{M0} =$ **879.1** kN

$N_{Ed} / N_{c,Rd} =$ **0.014**

PASS - Design compression resistance exceeds design compression

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

Critical buckling length — $L_{cr,y} = L_{m4_s2} =$ **2500** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr,y}^2 =$ **7760.4** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})} =$ **0.337**

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 — $\Phi_y = 0.5 \times (1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2) =$ **0.571**

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / (\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}), 1) =$ **0.969**

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} =$ **851.7** kN

$N_{Ed} / N_{b,y,Rd} =$ **0.015**

PASS - Design buckling resistance exceeds design compression

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

Critical buckling length — $L_{cr,z} = L_{m4_s2_seg1} =$ **2500** mm

Critical buckling force — $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr,z}^2 =$ **1020.1** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} =$ **0.928**

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 — $\Phi_z = 0.5 \times (1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2) =$ **1.055**

Buckling reduction factor - eq 6.49 — $\chi_z = \min(1 / (\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}), 1) =$ **0.643**

Design buckling resistance - eq 6.47 — $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} =$ **565.2** kN

$N_{Ed} / N_{b,z,Rd} =$ **0.022**

PASS - Design buckling resistance exceeds design compression

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length — $L_{cr,T} = L_{m4_s2_seg1_R} =$ **2500** 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 $i_0 = \sqrt{(i_y^2 + i_z^2)}$ = **91.0** mm

Elastic critical torsional buckling force $N_{cr,T} = (G \times I_t + \pi^2 \times E \times I_w / L_{cr,T}^2) / i_0^2$ = **1757.3** kN

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

Elastic critical torsional-flexural buckling force $N_{cr,TF}$ = **1757.3** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **1757.3** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{(A \times f_y / N_{cr})}$ = **0.707**

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 $\Phi_T = 0.5 \times (1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2)$ = **0.836**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / (\Phi_T + \sqrt{(\Phi_T^2 - \bar{\lambda}_T^2)}), 1)$ = **0.78**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **685.3** kN

$N_{Ed} / N_{b,T,Rd}$ = **0.018**

PASS - Design buckling resistance exceeds design compression

<u>Check design at start of span</u>

Check shear - Section 6.2.6

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

$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **6.2** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1282** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **203.5** kN

$V_{y,Ed} / V_{c,y,Rd}$ = **0.03**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed}$ = **15.4** kNm

Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **70.9** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.218**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 k_c = **0.77**

$C_1 = 1 / k_c^2$ = **1.687**

Poissons ratio ν = **0.3**

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

Unrestrained effective length $L = 1.0 \times L_{m4_s2_seg1_T}$ = **2500** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **205.5** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.587**

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 $\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2)$ = **0.661**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.923**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.895**

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 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **70.9** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.218**

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 $N_{y,lim} = \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t_w \times f_y / \gamma_{M0})$ = **147.0** kN

$N_{Ed} / N_{y,lim}$ = **0.086**

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 = 0 kNm / 15.429 kNm = **0.000**

α_y = 7.714 kNm / 15.429 kNm = **0.500**

$C_{my} = \max(0.6 + 0.4 \times \psi_y)$ = **0.600**

ψ_{LT} = 0 kNm / 15.429 kNm = **0.000**

α_{LT} = 7.714 kNm / 15.429 kNm = **0.500**

$C_{mLT} = \max(0.6 + 0.4 \times \psi_{LT})$ = **0.600**

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

Characteristic moment resistance $M_{y,Rk} = W_{pl.y} \times f_y$ = **70.9** kNm

Characteristic moment resistance $M_{z,Rk} = W_{pl.z} \times f_y$ = **19.5** kNm

Characteristic resistance to normal force $N_{Rk} = A \times f_y$ = **879.1** kN

Interaction factors $k_{yy} = C_{my} \times (1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}))$ = **0.601**

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

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.157**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1})$ = **0.257**

PASS - Combined bending and compression checks are satisfied