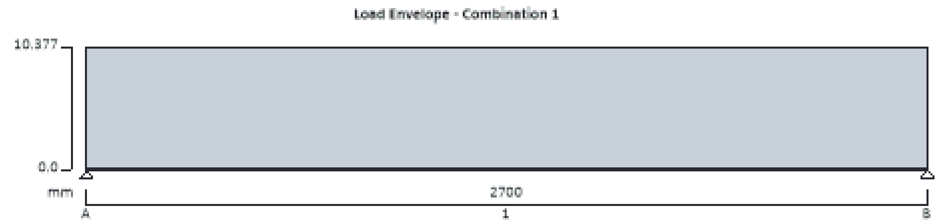
Appendix A

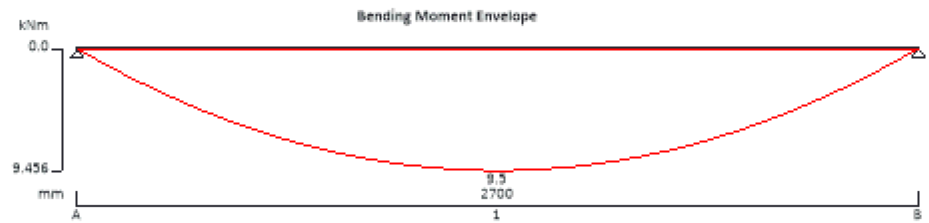
Haus Structures Consulting Structural Engineers Email: info@haus-structures.co.uk Website: www.haus-structures.co.uk	Project 92 Albert Road Horley Surrey RH6 7HZ				Job no 25039	
	Calcs for B1A - Beam				Start page no/Revision 1 A	
	Calc by AS	Calcs Date 7/1/2025	Checked by DG	Checked date July/2025	Approved by DG	Approved date July/2025

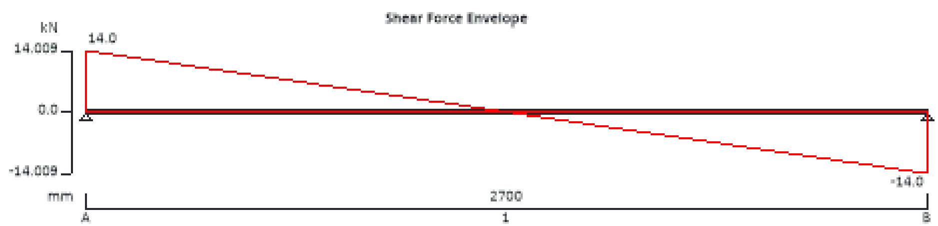
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Wall - Permanent full UDL 7.5 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50

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	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

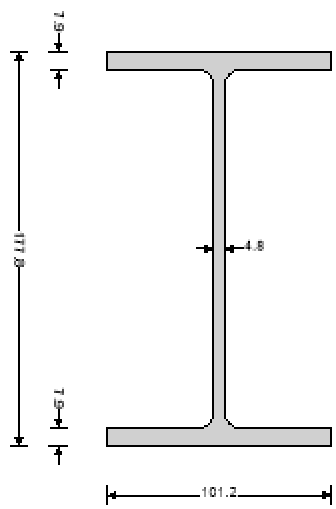
Maximum moment	M_{max} = **9.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **14** kN	V_{min} = **-14** kN
Deflection	δ_{max} = **1.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **14** kN	R_{A_min} = **14** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **10.4** kN	
Maximum reaction at support B	R_{B_max} = **14** kN	R_{B_min} = **14** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **10.4** kN	

Section details

Section type	**UB 178x102x19**
Steel grade	**S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **7.9** 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



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.000**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

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

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

Width of section — $c = d$ = **146.8** mm

$c / t_w = 37.6 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **40.6** mm

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

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **162** mm

Shear area factor — η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **14** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **985** mm²

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **201.9** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **9.5** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **60.8** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.948**

Poissons ratio — ν = **0.3**

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

Unrestrained length — $L = 1.0 \times L_{s1}$ = **2700** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **59.4** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.012**

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

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

Design resistance for buckling - 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 — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.988**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.692**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.973**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.712**

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **43.3** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = min(10 mm, L_{s1} / 500) = **5.4** mm

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **1.868** mm

PASS - Maximum deflection does not exceed deflection limit

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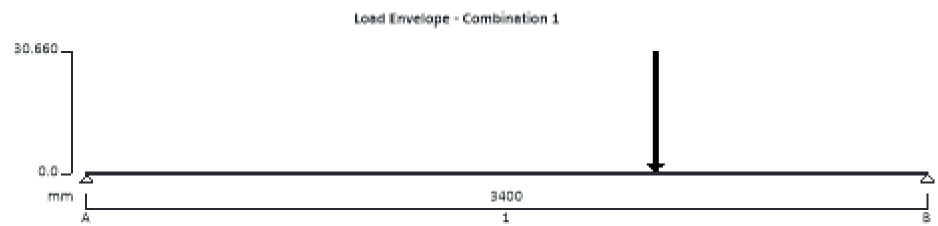
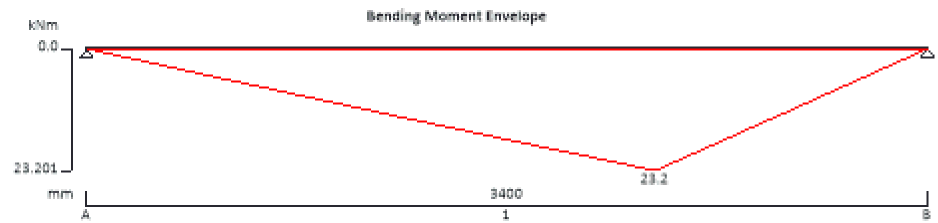
Email: info@haus-structures.co.uk Website: www.haus-structures.co.uk	Calc by AS	Calcs Date 7/1/2025	Checked by DG	Checked date July/2025	Approved by DG	Approved date July/2025

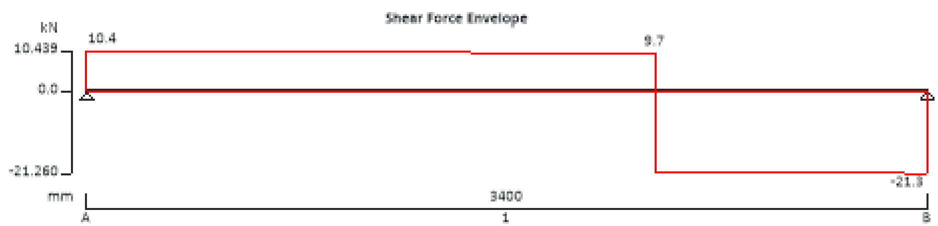
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Roof - Permanent point load 13.6 kN at 2300 mm
	Roof - Variable point load 8.2 kN at 2300 mm

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35

		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

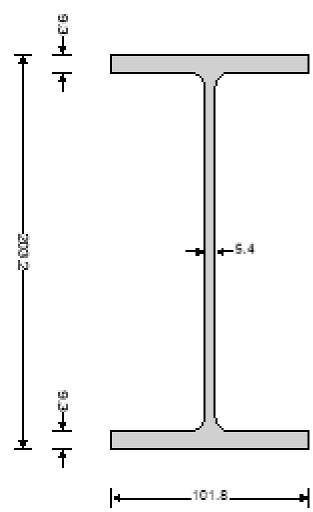
Maximum moment	M_{max} = **23.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **10.4** kN	V_{min} = **-21.3** kN
Deflection	δ_{max} = **3.5** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **10.4** kN	R_{A_min} = **10.4** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **4.8** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **2.7** kN	
Maximum reaction at support B	R_{B_max} = **21.3** kN	R_{B_min} = **21.3** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **9.6** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **5.5** kN	

Section details

Section type	**UB 203x102x23 (British Steel Section Range 2022 (BS4-1))**
Steel grade	**S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = max(t_f, t_w)$ = **9.3** 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



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	K_y = **1.000**

Effective length factor in minor axis — K_z = **1.000**

Effective length factor for torsion — $K_{LT.A}$ = **1.000**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

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

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

Width of section — $c = d$ = **169.4** mm

$c / t_w = 38.6 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **40.6** mm

$c / t_f = 5.4 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **184.6** mm

Shear area factor — η = **1.000**

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **21.3** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1238** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **253.7** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **23.2** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **83.1** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.96**

Poissons ratio — ν = **0.3**

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

Unrestrained length — $L = 1.0 \times L_{s1}$ = **3400** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **58.7** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.19**

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

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

Design resistance for buckling - 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 — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.166**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.585**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.979}$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.597}$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{49.6}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = \min(14\text{ mm}, L_{s1} / 250) = \mathbf{13.6}$ mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{3.496}$ mm

PASS - Maximum deflection does not exceed deflection limit

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Email: info@haus-structures.co.uk Website: www.haus-structures.co.uk	Calcs by DG	Calcs date 02/07/2025	Checked by DG	Checked date July 2025	Approved by DG	Approved date July 2025

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

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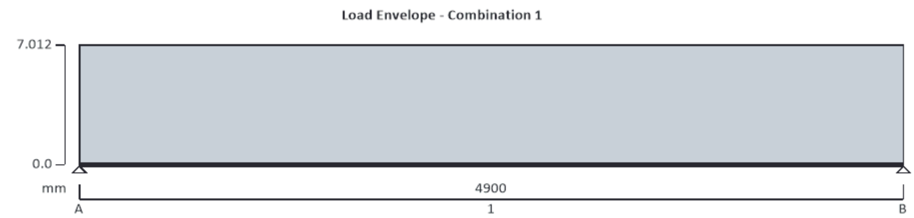
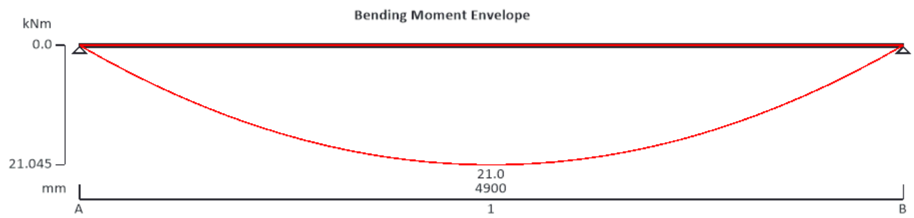
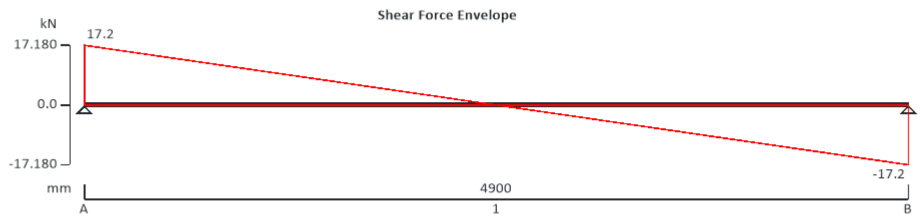
TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Roof - Permanent full UDL 2.4 kN/m
	Roof - Variable full UDL 2.25 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
		Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Maximum moment	M_{max} = **21** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **17.2** kN	V_{min} = **-17.2** kN
Deflection	δ_{max} = **6.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **17.2** kN	R_{A_min} = **17.2** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **6.6** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **5.5** kN	
Maximum reaction at support B	R_{B_max} = **17.2** kN	R_{B_min} = **17.2** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **6.6** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **5.5** kN	

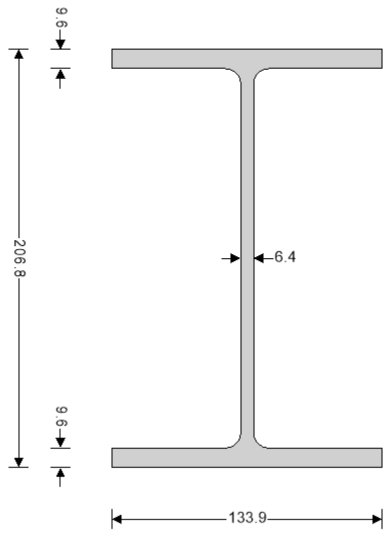
Section details

Section type **UB 203x133x30 (British Steel Section Range 2022 (BS4-1))**

Steel grade **S355**

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w)$ = **9.6** 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



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.200** + 2 × h
	$K_{LT.B}$ = **1.200** + 2 × h

Classification of cross sections - Section 5.5

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

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

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

$c / t_w = 33.1 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = \mathbf{56.2}$ mm

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

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = \mathbf{187.6}$ mm

Shear area factor $\eta = \mathbf{1.000}$

$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{17.2}$ kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1458}$ mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{298.7}$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{21}$ kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{111.6}$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 $k_c = \mathbf{0.94}$

$C_1 = 1 / k_c^2 = \mathbf{1.132}$

Curvature factor $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.931}$

Poissons ratio $\nu = \mathbf{0.3}$

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

Unrestrained length $L = 1.2 \times L_{s1} + 2 \times h = \mathbf{6294}$ mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} =$ **55.3** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{1.421}$

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

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5 b

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

Correction factor for rolled sections $\beta = \mathbf{0.75}$

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.431}$

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.463}$

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.993}$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.466}$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{52}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 500 =$ **9.8** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **6.103** mm

PASS - Maximum deflection does not exceed deflection limit