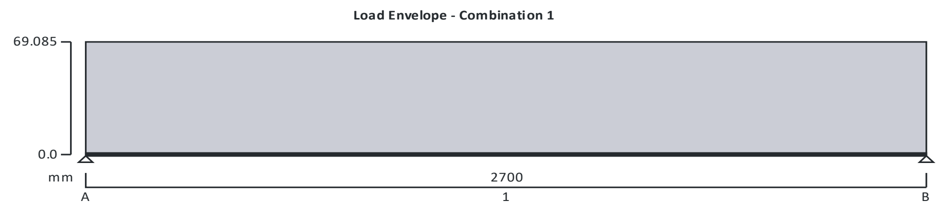
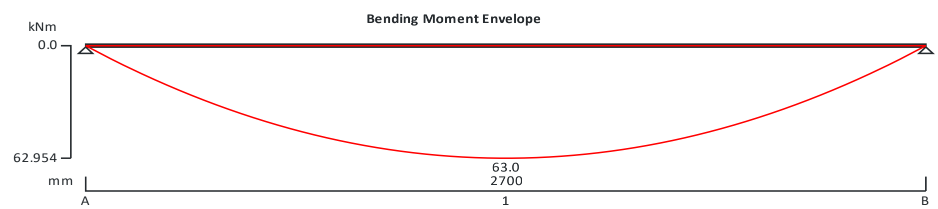
PlanningApplications.com Summer House, Upper Court Road Woldingham SURREY CR3 7BF support@planningapplications.com 07922	Project BEAM 1 203x203x46kgUC + 8x300mm s275 plate				Job no. 239	
	Calcs for Mr Simon Heath				Start page no./Revision 1	
	Calcs by SB	Calcs date 30/05/2023	Checked by DB	Checked date 30/05/2023	Approved by SB	Approved date 30/05/2023

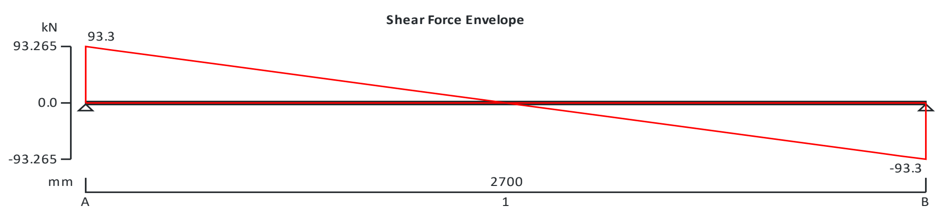
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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
	Permanent full UDL 28.5 kN/m
	Variable full UDL 20 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

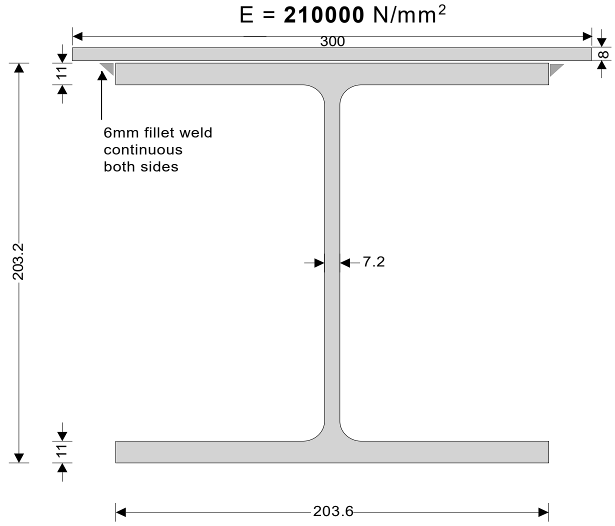
Maximum moment	M_{max} = **63** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **93.3** kN	V_{min} = **-93.3** kN
Deflection	δ_{max} = **1.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **93.3** kN	R_{A_min} = **93.3** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **39.1** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **27** kN	
Maximum reaction at support B	R_{B_max} = **93.3** kN	R_{B_min} = **93.3** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **39.1** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **27** kN	

Section details

Section type	**UC 203x203x46 (BS4-1)**
Steel grade	**S275**

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

Nominal thickness of element	$t = \max(t_f, t_w)$ = **11.0** 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



Flat 8x300mm plate, 2 No. 6mm fillet weld
Section depth, d, 300 mm
Section thickness, t, 8 mm
Mass of section, Mass, 18.8 kg/m
Area of section, A, 2400 mm^2
Radius of gyration about y-axis, i_y, 86.603 mm Radius of gyration about z-axis, i_z, 2.309 mm
Elastic section modulus about y-axis, $W_{el.y}$, 120000 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 3200 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 180000 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 4800 mm^3
Second moment of area about y-axis, I_y, 18000000 mm^4 Second moment of area about z-axis, I_z, 12800 mm^4

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

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

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

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

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t_w \times f_y) - (t_f + r)] / c, 1) = \mathbf{0.971}$

$c / t_w = 24.2 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 8.7 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{181.2}$ 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 — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{93.3}$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1698}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{269.5}$ kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{63}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{136.8}$ 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 — $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.813}$

Poissons ratio — $\nu = \mathbf{0.3}$

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

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

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{706.6}$ kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{0.44}$

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 — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{0.579}$

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.984}$

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.978}$

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{1.000}$

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{136.8}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check compression - Section 6.2.4

Design compression force — N_{Ed} = **300** kN

Design resistance of section - eq 6.10 — $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **1615.1** kN

Slenderness ratio for major (y-y) axis buckling

Critical buckling length — $L_{cr,y} = L_{s1} \times K_y$ = **2700** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E_{SEC3} \times I_y / L_{cr,y}^2$ = **12986.7** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{[A \times f_y / N_{cr,y}]}$ = **0.353**

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2 — b

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

Buckling reduction determination factor — $\phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2]$ = **0.588**

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / [\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}], 1)$ = **0.944**

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **1525.4** kN

PASS - Design buckling resistance exceeds design compression force

Slenderness ratio for minor (z-z) axis buckling

Critical buckling length — $L_{cr,z} = L_{s1\_seg1} \times K_z$ = **2700** mm

Critical buckling force — $N_{cr,z} = \pi^2 \times E_{SEC3} \times I_z / L_{cr,z}^2$ = **4401.7** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_z = \sqrt{[A \times f_y / N_{cr,z}]}$ = **0.606**

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2 — c

Imperfection factor - Table 6.1 — α_z = **0.49**

Buckling reduction determination factor — $\phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$ = **0.783**

Buckling reduction factor - eq 6.49 — $\chi_z = \min(1 / [\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}], 1)$ = **0.782**

Design buckling resistance - eq 6.47 — $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **1263** kN

PASS - Design buckling resistance exceeds design compression force

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length factor — K_T = **1.00**

Torsional buckling length — $L_{cr,T} = \max(L_{s1}, L_{s1\_seg1}) \times K_T$ = **2700** mm

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

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

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

Elastic critical torsional buckling force — $N_{cr,T} = 1 / i_0^2 \times [G \times I_t + \pi^2 \times E_{SEC3} \times I_w / L_{cr,T}^2]$ = **5621.9** kN

Torsion factor — $\beta_T = 1 - (y_0 / i_0)^2$ = **1.000**

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]]}$ = **5621.9** kN

Elastic critical buckling force — $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **5621.9** kN

Slenderness ratio for torsional buckling - eq 6.52 — $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]}$ = **0.536**

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2 — c

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

Buckling reduction determination factor — $\phi_T = 0.5 \times [1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2]$ = **0.726**

Buckling reduction factor - eq 6.49 — $\chi_T = \min(1 / [\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}], 1)$ = **0.823**

Design buckling resistance - eq 6.47 — $N_{b,T,Rd} = \chi_T \times A \times f_y / \gamma_{M1}$ = **1328.7** kN

PASS - Design buckling resistance exceeds design compression force

Combined bending and axial force - Section 6.2.9

Normal force to plastic resistance force ratio — $n = N_{Ed} / N_{pl,Rd} = \mathbf{0.19}$

Web area to gross area ratio — $a_w = \min((A - 2 \times b \times t_f) / A, 0.5) = \mathbf{0.24}$

Design plastic moment resistance - eq 6.13 — $M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{136.8}$ kNm

Reduced plastic moment resistance - eq 6.36 — $M_{N,Rd} = M_{pl,Rd} \times \min((1 - n) / (1 - 0.5 \times a_w), 1) = \mathbf{126.4}$ kNm

PASS - Reduced bending resistance moment exceeds design bending moment

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3

$M_{hy} = \mathbf{0}$ kNm

$M_{sy} = \mathbf{63}$ kNm

$\psi_y = \mathbf{1.000}$

$\alpha_{hy} = M_{hy} / M_{sy} = \mathbf{0.000}$

$C_{my} = 0.95 + 0.05 \times \alpha_{hy} = \mathbf{0.950}$

$M_{hz} = \mathbf{0}$ kNm

$M_{sz} = \mathbf{0}$ kNm

$\psi_z = \mathbf{1.000}$

$C_{mz} = 0.6 + 0.4 \times \psi_z = \mathbf{1.000}$

$M_{hLT} = \mathbf{0}$ kNm

$M_{sLT} = \mathbf{63}$ kNm

$\psi_{LT} = \mathbf{1.000}$

$\alpha_{hLT} = M_{hLT} / M_{sLT} = \mathbf{0.000}$

$C_{mLT} = 0.95 + 0.05 \times \alpha_{hLT} = \mathbf{0.950}$

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

Characteristic moment resistance — $M_{Rk} = W_{pl.y} \times f_y = \mathbf{136.8}$ kNm

Characteristic resistance to normal force — $N_{Rk} = A \times f_y = \mathbf{1615.1}$ kN

Interaction factors — $k_{yy} = C_{my} \times [1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})] = \mathbf{0.979}$

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) = \mathbf{0.979}$

Interaction formulae - eq 6.61 & eq 6.62 — $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1}) = \mathbf{0.654}$

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1}) = \mathbf{0.695}$

PASS - Combined bending and compression checks are satisfied

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360 = \mathbf{7.5}$ mm

Maximum deflection span 1 — $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min})) = \mathbf{1.443}$ mm

PASS - Maximum deflection does not exceed deflection limit