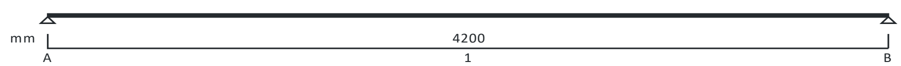
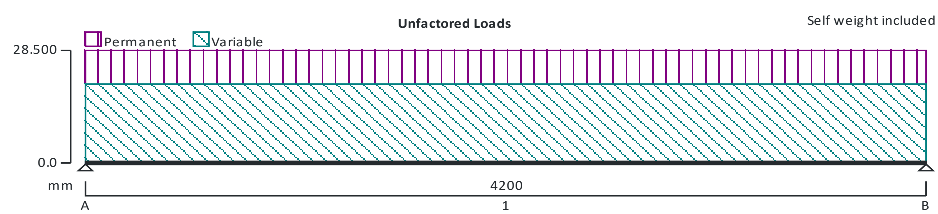
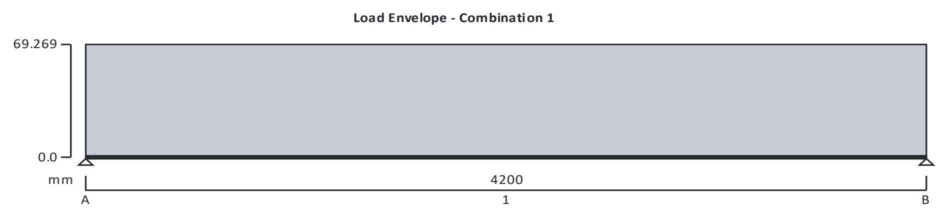
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

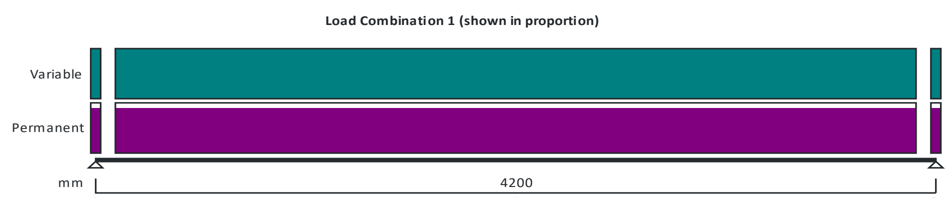
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

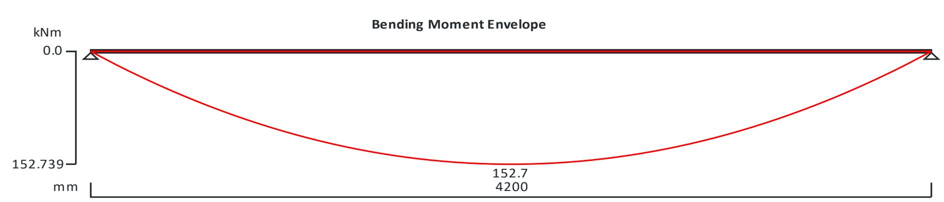
TEDDS calculation version 3.0.14

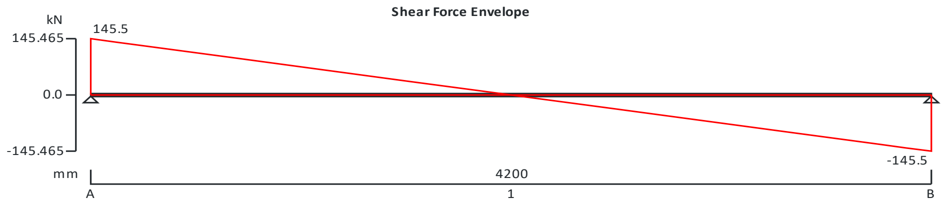












Support conditions

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} = **152.7** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **110.8** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **152.4** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum moment span 1 segment 3	$M_{s1\_seg3\_max}$ = **152.7** kNm	$M_{s1\_seg3\_min}$ = **0** kNm
Maximum moment span 1 segment 4	$M_{s1\_seg4\_max}$ = **124.7** kNm	$M_{s1\_seg4\_min}$ = **0** kNm
Maximum shear	V_{max} = **145.5** kN	V_{min} = **-145.5** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **145.5** kN	$V_{s1\_seg1\_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **76.2** kN	$V_{s1\_seg2\_min}$ = **0** kN
Maximum shear span 1 segment 3	$V_{s1\_seg3\_max}$ = **6.9** kN	$V_{s1\_seg3\_min}$ = **-62.3** kN
Maximum shear span 1 segment 4	$V_{s1\_seg4\_max}$ = **0** kN	$V_{s1\_seg4\_min}$ = **-145.5** kN
Deflection segment 5	δ_{max} = **6.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **145.5** kN	R_{A_min} = **145.5** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **61.1** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **42** kN	
Maximum reaction at support B	R_{B_max} = **145.5** kN	R_{B_min} = **145.5** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **61.1** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **42** kN	

Section details

Section type	**2 x UB 203x133x30 (BS4-1)**
Steel grade	**S275**

PlanningApplications.com Summer House, Upper Court Road Woldingham SURREY CR3 7BF support@planningapplications.com 07922 148 701	Project BEAM 2 (2 x 203x133x30kgUB) bolted togther min 1m spacing				Job no. 239	
	Calcs for Mr Simon Heath				Start page no./Revision 3	
	Calcs by SB	Calcs date 30/05/2023	Checked by DB	Checked date 30/05/2023	Approved by SB	Approved date 30/05/2023

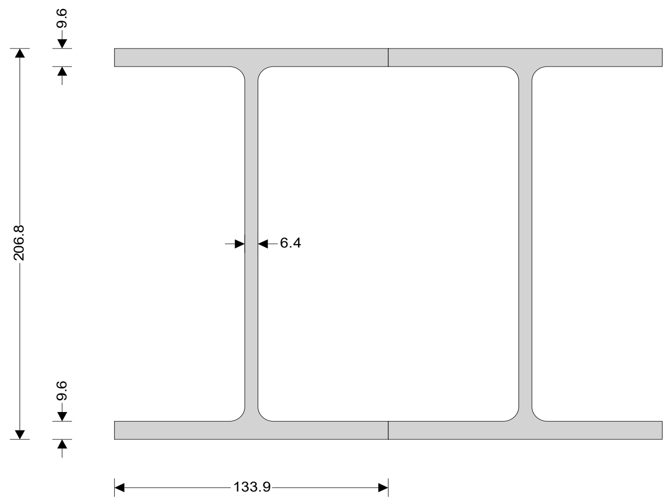
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element — $t = \max(t_f, t_w) = \mathbf{9.6}$ mm

Nominal yield strength — $f_y = \mathbf{275}$ N/mm^2

Nominal ultimate tensile strength — $f_u = \mathbf{410}$ N/mm^2

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



Partial factors - Section 6.1

Resistance of cross-sections — $\gamma_{M0} = \mathbf{1.00}$

Resistance of members to instability — $\gamma_{M1} = \mathbf{1.00}$

Resistance of tensile members to fracture — $\gamma_{M2} = \mathbf{1.10}$

Lateral restraint

Both beams bolted/restraint together, bolts located min distances plus 1000 mm, 2000 mm and 3000mm. M16 bolts.

Effective length factors

Effective length factor in major axis — $K_y = \mathbf{1.000}$

Effective length factor in minor axis — $K_z = \mathbf{1.000}$

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

$K_{LT.B} = \mathbf{1.000}$

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 — $c = d = \mathbf{172.4}$ mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times N \times t_w \times f_y) - (t_f + r)] / c, 1) = \mathbf{0.747}$

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

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{56.2}$ 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 — $h_w = h - 2 \times t_f = \mathbf{187.6}$ mm

Shear area factor — η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **145.5** kN

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

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **462.8** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 1 segment 3 major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_seg3\_max}), \text{abs}(M_{s1\_seg3\_min}))$ = **152.7** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl.y} \times f_y / \gamma_{M0}$ = **172.9** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.978**

$C_1 = 1 / k_c^2$ = **1.046**

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

Poissons ratio — ν = **0.3**

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

Unrestrained length — $L = 1.0 \times L_{s1\_seg3}$ = **1000** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **929.4** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.305**

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

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can 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 — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.519**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.994**

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

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times N \times W_{pl.y} \times f_y / \gamma_{M1}$ = **172.9** 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 — $N_{c,Rd} = N_{pl,Rd} = N \times A \times f_y / \gamma_{M0}$ = **2101.6** kN

Slenderness ratio for major (y-y) axis buckling

Critical buckling length — $L_{cr,y} = L_{s1} \times K_y$ = **4200** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E_{SEC3} \times I_y / L_{cr,y}^2$ = **3402.2** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{[A \times f_y / N_{cr,y}]}$ = **0.556**

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2 — a

Imperfection factor - Table 6.1 — α_y = **0.21**

Buckling reduction determination factor — $\phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2]$ = **0.692**

Buckling reduction factor - eq 6.49 — $\chi_y = \min(1 / [\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}], 1)$ = **0.906**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times N \times A \times f_y / \gamma_{M1}$ = **1904.1** kN

PASS - Design buckling resistance exceeds design compression force

Slenderness ratio for minor (z-z) axis buckling

Critical buckling length $L_{cr,z} = L_{s1\_seg3} \times K_z$ = **1000** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E_{SEC3} \times I_z / L_{cr,z}^2$ = **7972.4** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{[A \times f_y / N_{cr,z}]}$ = **0.363**

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2 b

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

Buckling reduction determination factor $\phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$ = **0.594**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / [\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}], 1)$ = **0.940**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times N \times A \times f_y / \gamma_{M1}$ = **1976.5** 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 $L_{cr,T} = \max(L_{s1}, L_{s1\_seg3}) \times K_T$ = **4200** mm

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

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

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

Elastic critical torsional buckling force $N_{cr,T} = 1 / i_0^2 \times [G \times I_t + \pi^2 \times E_{SEC3} \times I_w / L_{cr,T}^2]$ = **1481.4** kN

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

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}]$ = **1481.4** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **1481.4** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]}$ = **0.842**

Design resistance for 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 $\phi_T = 0.5 \times [1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2]$ = **0.964**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / [\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}], 1)$ = **0.698**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times N \times A \times f_y / \gamma_{M1}$ = **1467.1** kN

PASS - Design buckling resistance exceeds design compression force

Combined bending and axial force - Section 6.2.9

Bending and axial force check - eq 6.33 & 6.34 $N_{Ed} <= \min(0.25 \times N_{pl,Rd}, 0.5 \times N \times h_w \times t_w \times f_y / \gamma_{M0})$

No allowance on the plastic moment need to be accounted for due to the effect of axial force

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 M_{hy} = **0** kNm

M_{sy} = **153** kNm

ψ_y = **1.000**

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

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

M_{hz} = **0** kNm

M_{sz} = **0** kNm

ψ_z = **1.000**

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

$M_{hLT} =$ **152** kNm

$M_{sLT} =$ **147** kNm

$\psi_{LT} =$ **0.818**

$\alpha_{sLT} = M_{sLT} / M_{hLT} =$ **0.966**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{sLT}, 0.4) =$ **0.973**

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

Characteristic moment resistance — $M_{Rk} = N \times W_{pl.y} \times f_y =$ **172.9** kNm

Characteristic resistance to normal force — $N_{Rk} = N \times A \times f_y =$ **2101.6** kN

Interaction factors — $k_{yy} = C_{my} \times [1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})] =$ **1.003**

$k_{zy} = \min(0.6 + \bar{\lambda}_z, 1 - 0.1 \times \bar{\lambda}_z \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})) =$ **0.963**

Interaction formulae - eq 6.61 & eq 6.62 — $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1}) =$ **1.044**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1}) =$ **1.003**

FAIL - Combined bending and compression checks are not satisfied

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360 =$ **11.7** mm

Maximum deflection span 1 — $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **6.663** mm

PASS - Maximum deflection does not exceed deflection limit