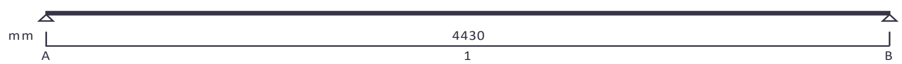
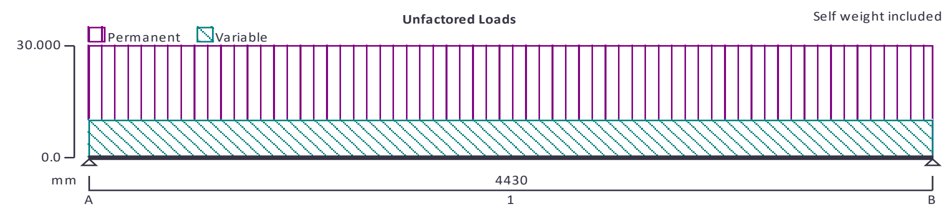
PlanningApplications.com Summer House, Upper Court Road Woldingham SURREY CR3 7BF support@planningapplications.com 07922 148 701	Project BEAM 4 (2 No. 203x133x25kgUB) bolted together min 0.4m				Job no. 2023-7459	
	Calcs for Safdar Zia - 313 Sandy Lane Manchester				Start page no./Revision 1	
	Calcs by SB	Calcs date 17/09/2023	Checked by DB	Checked date 17/09/2023	Approved by SB	Approved date 17/09/2023

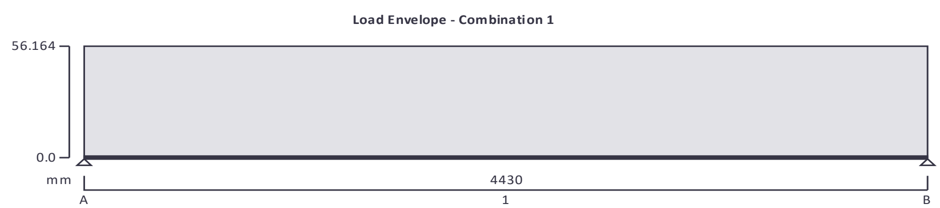
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

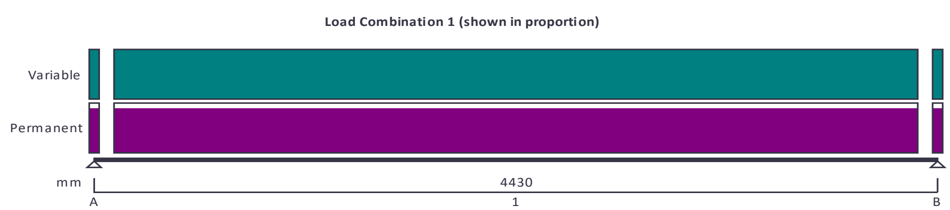
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

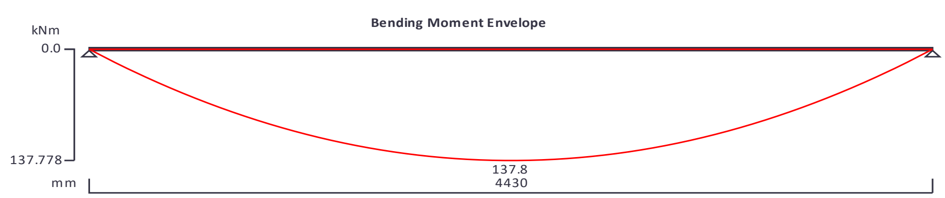
TEDDS calculation version 3.0.14

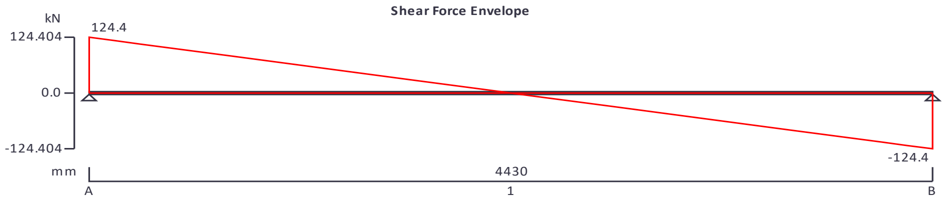












Support conditions

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 30 kN/m
	Variable full UDL 10 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} = **137.8** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **102.9** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **137.8** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum moment span 1 segment 3	$M_{s1\_seg3\_max}$ = **137.8** kNm	$M_{s1\_seg3\_min}$ = **0** kNm
Maximum moment span 1 segment 4	$M_{s1\_seg4\_max}$ = **104.7** kNm	$M_{s1\_seg4\_min}$ = **0** kNm
Maximum shear	V_{max} = **124.4** kN	V_{min} = **-124.4** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **124.4** kN	$V_{s1\_seg1\_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **62.6** kN	$V_{s1\_seg2\_min}$ = **0** kN
Maximum shear span 1 segment 3	$V_{s1\_seg3\_max}$ = **0.8** kN	$V_{s1\_seg3\_min}$ = **-60.9** kN
Maximum shear span 1 segment 4	$V_{s1\_seg4\_max}$ = **0** kN	$V_{s1\_seg4\_min}$ = **-124.4** kN
Deflection segment 5	δ_{max} = **5.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **124.4** kN	R_{A_min} = **124.4** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **67.5** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **22.1** kN	
Maximum reaction at support B	R_{B_max} = **124.4** kN	R_{B_min} = **124.4** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **67.5** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **22.2** kN	

Section details

Section type	**2 x UB 203x133x25 (BS4-1)**
Steel grade	**S275**

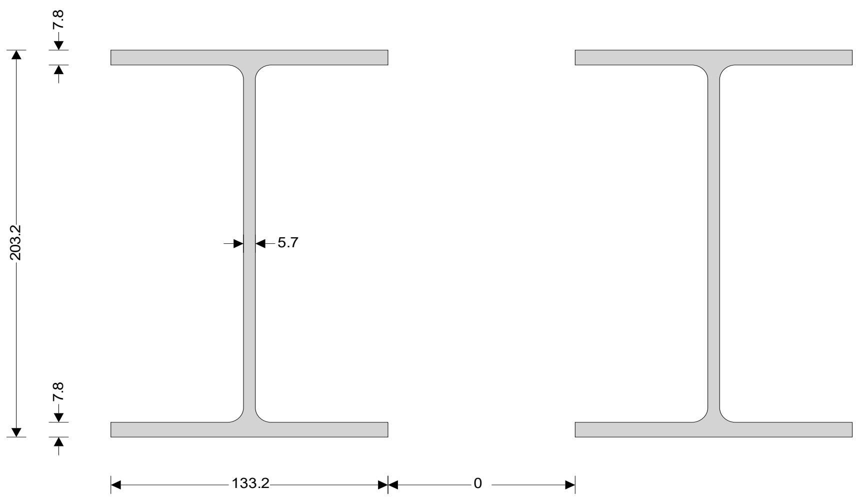
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = \max(t_f, t_w) = \mathbf{7.8}$ 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

Span 1 has lateral restraint at supports plus 1100 mm, 2200 mm and 3300 mm

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

$\alpha = \min([h / 2 + N_{Ed} / (2 \times N \times t_w \times f_y) - (t_f + r)] / c, 1) = \mathbf{0.778}$

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

$c / t_f = 7.8 \times \varepsilon <= 9 \times \varepsilon$ 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 — $V_{Ed} = max(abs(V_{max}), abs(V_{min}))$ = **124.4** kN

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

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **407.1** 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 — $M_{Ed} = max(abs(M_{s1\_seg3\_max}), abs(M_{s1\_seg3\_min}))$ = **137.8** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = N \times W_{pl.y} \times f_y / \gamma_{M0}$ = **141.8** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.971**

$C_1 = 1 / k_c^2$ = **1.062**

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.932**

Poissons ratio — ν = **0.3**

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

Unrestrained length — $L = 1.0 \times L_{s1\_seg3}$ = **1100** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **613.8** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.34**

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 — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.533**

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 — $f = min(1 - 0.5 \times (1 - k_c)\times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.992**

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

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times N \times W_{pl.y} \times f_y / \gamma_{M1}$ = **141.8** 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 — $N_{c,Rd} = N_{pl,Rd} = N \times A \times f_y / \gamma_{M0}$ = **1758.3** kN

Slenderness ratio for major (y-y) axis buckling

Critical buckling length — $L_{cr,y} = L_{s1} \times K_y$ = **4430** mm

Critical buckling force — $N_{cr,y} = \pi^2 \times E_{SEC3} \times I_y / L_{cr,y}^2$ = **2471.5** kN

Slenderness ratio for buckling - eq 6.50 — $\bar{\lambda}_y = \sqrt{[A \times f_y / N_{cr,y}]}$ = **0.596**

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 — $\phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2]$ = **0.719**

Buckling reduction factor - eq 6.49 — $\chi_y = min(1 / [\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}], 1)$ = **0.891**

Design buckling resistance - eq 6.47 — $N_{b,y,Rd} = \chi_y \times N \times A \times f_y / \gamma_{M1} =$ **1567.2** kN

PASS - Design buckling resistance exceeds design compression force

Slenderness ratio for minor (z-z) axis buckling

Critical buckling length — $L_{cr,z} = L_{s1\_seg3} \times K_z =$ **1100** mm

Critical buckling force — $N_{cr,z} = \pi^2 \times E_{SEC3} \times I_z / L_{cr,z}^2 =$ **5269.4** kN

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

Design resistance for buckling - 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 — $N_{b,z,Rd} = \chi_z \times N \times A \times f_y / \gamma_{M1} =$ **1622.3** 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 — $L_{cr,T} = \max(L_{s1}, L_{s1\_seg3}) \times K_T =$ **4430** 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 — $N_{cr,T} = 1 / i_0^2 \times [G \times I_t + \pi^2 \times E_{SEC3} \times I_w / L_{cr,T}^2] =$ **956** kN

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

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}] =$ **956** kN

Elastic critical buckling force — $N_{cr} = \min(N_{cr,T}, N_{cr,TF}) =$ **956** kN

Slenderness ratio for torsional buckling - eq 6.52 — $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]} =$ **0.959**

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2 — b

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

Buckling reduction determination factor — $\phi_T = 0.5 \times [1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2] =$ **1.089**

Buckling reduction factor - eq 6.49 — $\chi_T = \min(1 / [\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}], 1) =$ **0.623**

Design buckling resistance - eq 6.47 — $N_{b,T,Rd} = \chi_T \times N \times A \times f_y / \gamma_{M1} =$ **1095.8** 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 — $n = N_{Ed} / N_{pl,Rd} =$ **0.17**

Web area to gross area ratio — $a_w = \min((A - 2 \times b \times t_f) / A, 0.5) =$ **0.35**

Design plastic moment resistance - eq 6.13 — $M_{pl,Rd} = N \times W_{pl.y} \times f_y / \gamma_{M0} =$ **141.8** kNm

Reduced plastic moment resistance - eq 6.36 — $M_{N,Rd} = M_{pl,Rd} \times \min((1 - n) / (1 - 0.5 \times a_w), 1) =$ **141.8** 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 — $M_{hy} =$ **0** kNm

$M_{sy} =$ **138** kNm

$\psi_y =$ **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} = **138** kNm

M_{sLT} = **130** kNm

ψ_{LT} = **0.760**

$\alpha_{sLT} = M_{sLT} / M_{hLT}$ = **0.942**

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{sLT}, 0.4)$ = **0.953**

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

Characteristic moment resistance — $M_{Rk} = N \times W_{pl.y} \times f_y$ = **141.8** kNm

Characteristic resistance to normal force — $N_{Rk} = N \times A \times f_y$ = **1758.3** kN

Interaction factors — $k_{yy} = C_{my} \times [1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})]$ = **1.022**

$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})$ = **0.989**

Interaction formulae - eq 6.61 & eq 6.62 — $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1})$ = **1.185**

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1})$ = **1.146**

FAIL - Combined bending and compression checks are not satisfied

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360$ = **12.3** mm

Maximum deflection span 1 — $\delta = \max(abs(\delta_{max}), abs(\delta_{min}))$ = **5.102** mm

PASS - Maximum deflection does not exceed deflection limit