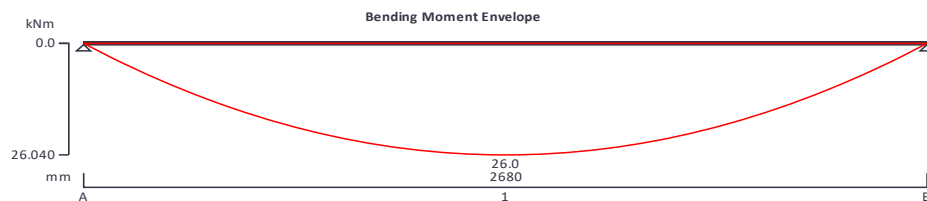
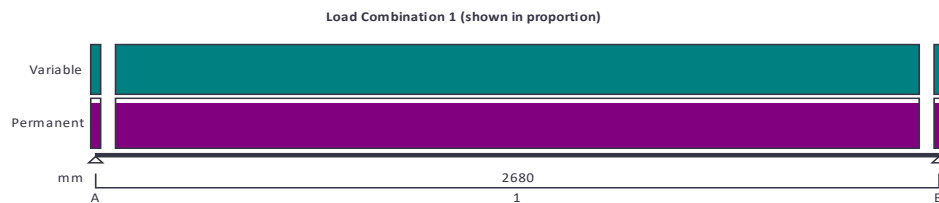
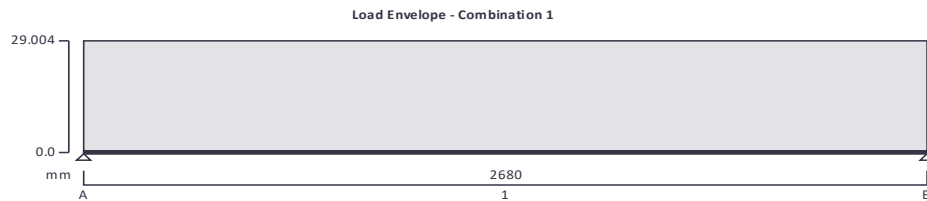
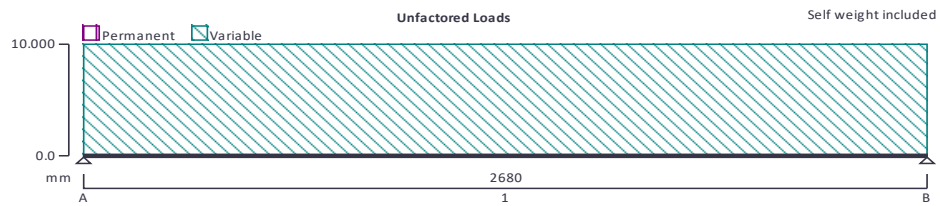


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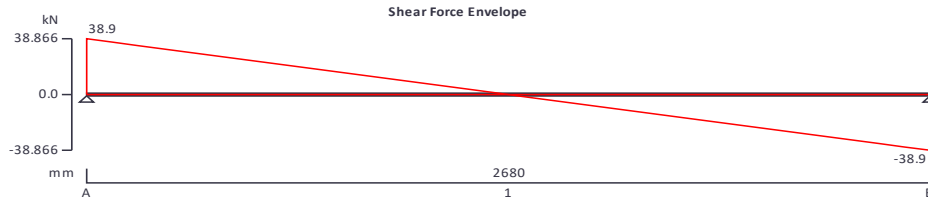
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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



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Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam \times 1 Permanent full UDL 10 kN/m Variable full UDL 10 kN/m
------------	---

Load combinations

Load combination 1	Support A	Permanent \times 1.35 Variable \times 1.50 Permanent \times 1.35 Variable \times 1.50
	Support B	Permanent \times 1.35 Variable \times 1.50

Analysis results

Maximum moment	$M_{max} = 26 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum moment span 1 segment 1	$M_{s1_seg1_max} = 12.5 \text{ kNm}$	$M_{s1_seg1_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 2	$M_{s1_seg2_max} = 25.4 \text{ kNm}$	$M_{s1_seg2_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 3	$M_{s1_seg3_max} = 26 \text{ kNm}$	$M_{s1_seg3_min} = 0 \text{ kNm}$
Maximum moment span 1 segment 4	$M_{s1_seg4_max} = 14 \text{ kNm}$	$M_{s1_seg4_min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 38.9 \text{ kN}$	$V_{min} = -38.9 \text{ kN}$
Maximum shear span 1 segment 1	$V_{s1_seg1_max} = 38.9 \text{ kN}$	$V_{s1_seg1_min} = 0 \text{ kN}$
Maximum shear span 1 segment 2	$V_{s1_seg2_max} = 28 \text{ kN}$	$V_{s1_seg2_min} = 0 \text{ kN}$
Maximum shear span 1 segment 3	$V_{s1_seg3_max} = 6.2 \text{ kN}$	$V_{s1_seg3_min} = -26.4 \text{ kN}$
Maximum shear span 1 segment 4	$V_{s1_seg4_max} = 0 \text{ kN}$	$V_{s1_seg4_min} = -38.9 \text{ kN}$
Deflection segment 5	$\delta_{max} = 1.2 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 38.9 \text{ kN}$	$R_{A_min} = 38.9 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 13.9 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 13.4 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 38.9 \text{ kN}$	$R_{B_min} = 38.9 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 13.9 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 13.4 \text{ kN}$	

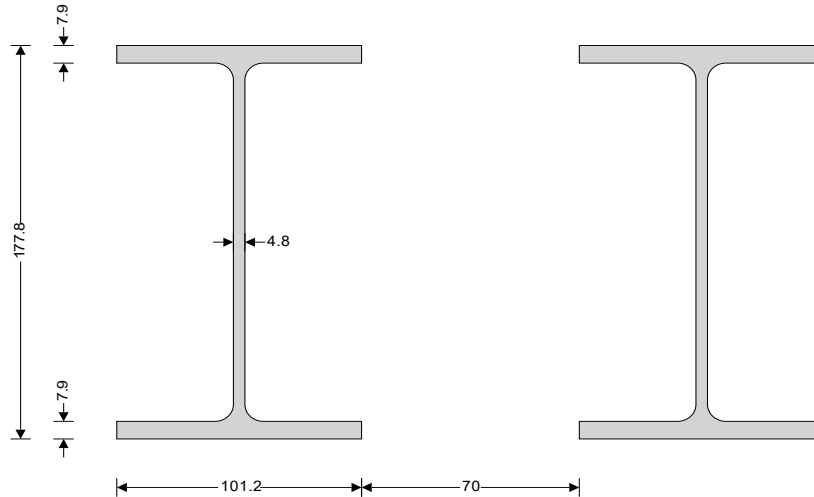
Section details

Section type	2 x UB 178x102x19
Steel grade	S275

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

Nominal thickness of element $t = \max(t_f, t_w) = 7.9 \text{ mm}$
 Nominal yield strength $f_y = 275 \text{ N/mm}^2$
 Nominal ultimate tensile strength $f_u = 410 \text{ N/mm}^2$
 Modulus of elasticity $E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections $\gamma_{M0} = 1.00$
 Resistance of members to instability $\gamma_{M1} = 1.00$
 Resistance of tensile members to fracture $\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports plus 375 mm, 1125 mm and 2250 mm

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

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

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

Width of section $c = d = 146.8 \text{ mm}$
 $\alpha = \min([h / 2 + N_{Ed} / (2 \times N \times t_w \times f_y) - (t_f + r)] / c, 1) = 0.887$
 $c / t_w = 33.1 \times \varepsilon \leq 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = 40.6 \text{ mm}$
 $c / t_f = 5.6 \times \varepsilon \leq 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = 162 \text{ mm}$

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Shear area factor

$$\eta = 1.000$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 38.9 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 985 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 312.8 \text{ kN}$$

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

$$M_{Ed} = \max(\text{abs}(M_{s1_seg3_max}), \text{abs}(M_{s1_seg3_min})) = 26 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = N \times W_{pl,y} \times f_y / \gamma_{M0} = 94.2 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.961$$

$$C_1 = 1 / k_c^2 = 1.084$$

Curvature factor

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

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained length

$$L = 1.0 \times L_{s1_seg3} = 1125 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 240.1 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.443$$

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

$$\alpha_{LT} = 0.34$$

Correction factor for rolled sections

$$\beta = 0.75$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.581$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.983$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.985$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.998$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times N \times W_{pl,y} \times f_y / \gamma_{M1} = 94 \text{ kNm}$$

PASS - Design buckling resistance moment exceeds design bending moment

Check compression - Section 6.2.4

Design compression force

$$N_{Ed} = 300 \text{ kN}$$

Design resistance of section - eq 6.10

$$N_{c,Rd} = N_{pl,Rd} = N \times A \times f_y / \gamma_{M0} = 1334.4 \text{ kN}$$

Slenderness ratio for major (y-y) axis buckling

Critical buckling length

$$L_{cr,y} = L_{s1} \times K_y = 2680 \text{ mm}$$

Critical buckling force

$$N_{cr,y} = \pi^2 \times E_{SEC3} \times I_y / L_{cr,y}^2 = 3912.9 \text{ kN}$$

Slenderness ratio for buckling - eq 6.50

$$\bar{\lambda}_y = \sqrt{[A \times f_y / N_{cr,y}]} = 0.413$$

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2

$$a$$

Imperfection factor - Table 6.1

$$\alpha_y = 0.21$$

Buckling reduction determination factor

$$\phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = 0.608$$

Buckling reduction factor - eq 6.49

$$\chi_y = \min(1 / [\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}], 1) = 0.949$$

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Design buckling resistance - eq 6.47

$$N_{b,y,Rd} = \chi_y \times N \times A \times f_y / \gamma_{M1} = \mathbf{1266.8 \text{ kN}}$$

PASS - Design buckling resistance exceeds design compression force

Slenderness ratio for minor (z-z) axis buckling

Critical buckling length

$$L_{cr,z} = L_{s1_seg3} \times K_z = \mathbf{1125 \text{ mm}}$$

Critical buckling force

$$N_{cr,z} = \pi^2 \times E_{SEC3} \times I_z / L_{cr,z}^2 = \mathbf{2238.7 \text{ kN}}$$

Slenderness ratio for buckling - eq 6.50

$$\bar{\lambda}_z = \sqrt{[A \times f_y / N_{cr,z}]} = \mathbf{0.546}$$

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2

b

Imperfection factor - Table 6.1

$$\alpha_z = \mathbf{0.34}$$

Buckling reduction determination factor

$$\phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] = \mathbf{0.708}$$

Buckling reduction factor - eq 6.49

$$\chi_z = \min(1 / [\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}], 1) = \mathbf{0.863}$$

Design buckling resistance - eq 6.47

$$N_{b,z,Rd} = \chi_z \times N \times A \times f_y / \gamma_{M1} = \mathbf{1152 \text{ kN}}$$

PASS - Design buckling resistance exceeds design compression force

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length factor

$$K_T = \mathbf{1.00}$$

Torsional buckling length

$$L_{cr,T} = \max(L_{s1}, L_{s1_seg3}) \times K_T = \mathbf{2680 \text{ mm}}$$

Distance from shear centre to centroid in y axis

$$y_0 = \mathbf{0.0 \text{ mm}}$$

Distance from shear centre to centroid in z axis

$$z_0 = \mathbf{0.0 \text{ mm}}$$

Radius of gyration

$$i_0 = \sqrt{[i_y^2 + i_z^2]} = \mathbf{78.4 \text{ mm}}$$

Elastic critical torsional buckling force

$$N_{cr,T} = 1 / i_0^2 \times [G \times I_t + \pi^2 \times E_{SEC3} \times I_w / L_{cr,T}^2] = \mathbf{1041.4 \text{ kN}}$$

Torsion factor

$$\beta_T = 1 - (y_0 / i_0)^2 = \mathbf{1.000}$$

Elastic critical torsional-flexural buckling force

$$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}] = \mathbf{1041.4 \text{ kN}}$$

Elastic critical buckling force

$$N_{cr} = \min(N_{cr,T}, N_{cr,TF}) = \mathbf{1041.4 \text{ kN}}$$

Slenderness ratio for torsional buckling - eq 6.52

$$\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]} = \mathbf{0.800}$$

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2

b

Imperfection factor - Table 6.1

$$\alpha_T = \mathbf{0.34}$$

Buckling reduction determination factor

$$\phi_T = 0.5 \times [1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2] = \mathbf{0.922}$$

Buckling reduction factor - eq 6.49

$$\chi_T = \min(1 / [\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}], 1) = \mathbf{0.724}$$

Design buckling resistance - eq 6.47

$$N_{b,T,Rd} = \chi_T \times N \times A \times f_y / \gamma_{M1} = \mathbf{966.4 \text{ kN}}$$

PASS - Design buckling resistance exceeds design compression force

Combined bending and axial force - Section 6.2.9

Normal force to plastic resistance force ratio

$$n = N_{Ed} / N_{pl,Rd} = \mathbf{0.22}$$

Web area to gross area ratio

$$a_w = \min((A - 2 \times b \times t_f) / A, 0.5) = \mathbf{0.34}$$

Design plastic moment resistance - eq 6.13

$$M_{pl,Rd} = N \times W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{94.2 \text{ kNm}}$$

Reduced plastic moment resistance - eq 6.36

$$M_{N,Rd} = M_{pl,Rd} \times \min((1 - n) / (1 - 0.5 \times a_w), 1) = \mathbf{88.0 \text{ kNm}}$$

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

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

$$M_{sy} = \mathbf{26 \text{ kNm}}$$

$$\psi_y = \mathbf{1.000}$$

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

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

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

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

$$\psi_z = 1.000$$

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

$$M_{hLT} = 25 \text{ kNm}$$

$$M_{sLT} = 24 \text{ kNm}$$

$$\psi_{LT} = 0.553$$

$$\alpha_{sLT} = M_{sLT} / M_{hLT} = 0.957$$

$$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{sLT}, 0.4) = 0.966$$

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

Characteristic moment resistance

$$M_{Rk} = N \times W_{ply} \times f_y = 94.2 \text{ kNm}$$

Characteristic resistance to normal force

$$N_{Rk} = N \times A \times f_y = 1334.4 \text{ kN}$$

Interaction factors

$$k_{yy} = C_{my} \times [1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})] = 0.998$$

$$k_{zy} = 1 - 0.1 \times \min(1, \bar{\lambda}_z) \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1}) = 0.980$$

Interaction formulae - eq 6.61 & eq 6.62

$$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1}) = 0.517$$

$$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1}) = 0.536$$

PASS - Combined bending and compression checks are satisfied

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 7.4 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 1.179 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit