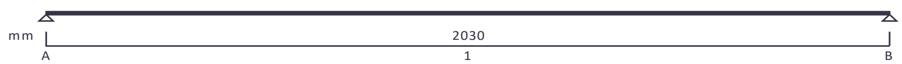
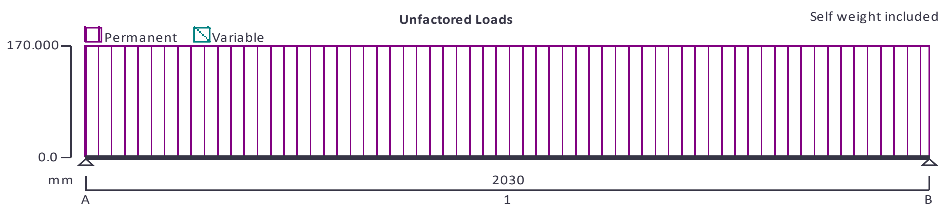
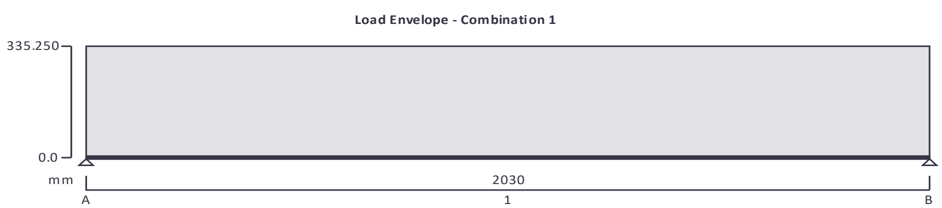
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

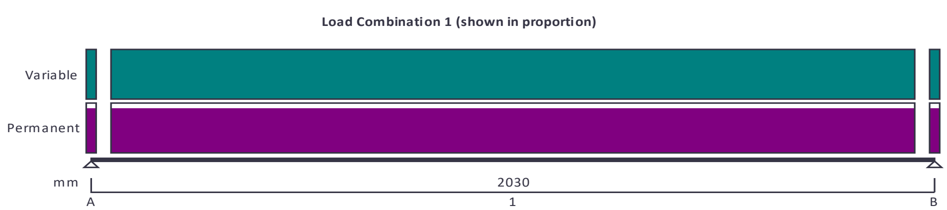
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

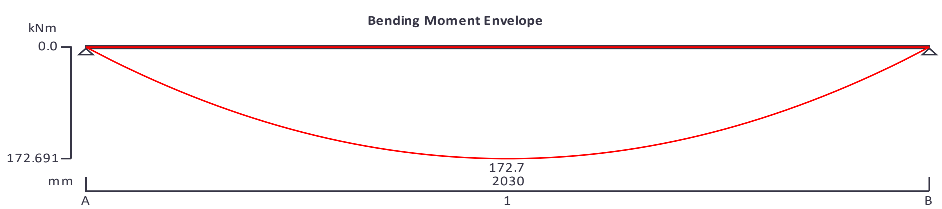
TEDDS calculation version 3.0.14

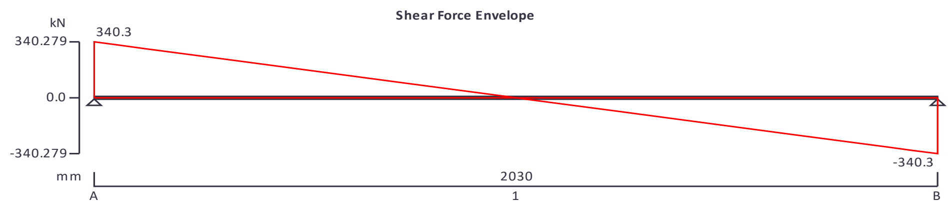












Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Variable full UDL 70 kN/m
	Permanent full UDL 170 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} = **172.7** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **128.2** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **172.7** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum moment span 1 segment 3	$M_{s1\_seg3\_max}$ = **172.7** kNm	$M_{s1\_seg3\_min}$ = **0** kNm
Maximum moment span 1 segment 4	$M_{s1\_seg4\_max}$ = **133.3** kNm	$M_{s1\_seg4\_min}$ = **0** kNm
Maximum shear	V_{max} = **340.3** kN	V_{min} = **-340.3** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **340.3** kN	$V_{s1\_seg1\_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **172.7** kN	$V_{s1\_seg2\_min}$ = **0** kN
Maximum shear span 1 segment 3	$V_{s1\_seg3\_max}$ = **5** kN	$V_{s1\_seg3\_min}$ = **-162.6** kN
Maximum shear span 1 segment 4	$V_{s1\_seg4\_max}$ = **0** kN	$V_{s1\_seg4\_min}$ = **-340.3** kN
Deflection segment 5	δ_{max} = **0.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **340.3** kN	R_{A_min} = **340.3** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **173.1** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **71.1** kN	
Maximum reaction at support B	R_{B_max} = **340.3** kN	R_{B_min} = **340.3** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **173.1** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **71.1** kN	

Section details

Section type	**2 x UB 254x102x28 (BS4-1)**
Steel grade	**S275**

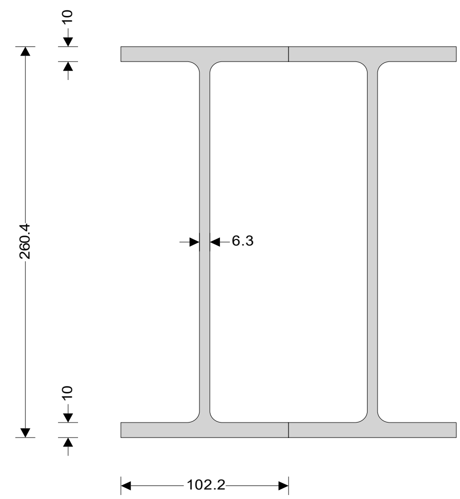
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = max(t_f, t_w) = \mathbf{10.0}$ 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 500 mm, 1000 mm and 1500 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 - Table 5.2 (sheet 1 of 3)

Width of section $c = d = \mathbf{225.2}$ mm

$c / t_w = 38.7 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = \mathbf{40.3}$ mm

$c / t_f = 4.4 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = \mathbf{240.4}$ 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	$V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{340.3}$ kN
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1779}$ mm^2
Design shear resistance - cl 6.2.6(2)	$V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{564.9}$ kN

PASS - Design shear resistance exceeds design shear force

Combined bending and shear - Section 6.2.8

Reduction factor - cl.6.2.8(3)	$\rho_v = [(2 \times V_{Ed} / V_{pl,Rd}) - 1]^2 = \mathbf{0.042}$

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

Design bending moment	$M_{Ed} = \max(\text{abs}(M_{s1\_seg3\_max}), \text{abs}(M_{s1\_seg3\_min})) = \mathbf{172.7}$ kNm
Design bending resistance moment - eq 6.13	$M_{c,Rd} = M_{pl,Rd} = N \times [(W_{pl.y} - t_w \times h^2 / 4) + (t_w \times h^2 / 4) \times (1 - \rho_v)] \times f_y / \gamma_{M0} = \mathbf{191.6}$ kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	$k_c = \mathbf{0.972}$
	$C_1 = 1 / k_c^2 = \mathbf{1.058}$
Curvature factor	$g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.977}$
Poissons ratio	$\nu = \mathbf{0.3}$
Shear modulus	$G = E / [2 \times (1 + \nu)] = \mathbf{80769}$ N/mm^2
Unrestrained length	$L = 1.0 \times L_{s1\_seg3} = \mathbf{500}$ mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{2038.6}$ kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{([(W_{pl.y} - t_w \times h^2 / 4) + (t_w \times h^2 / 4) \times (1 - \rho_v)] \times f_y / M_{cr})} = \mathbf{0.217}$
Limiting slenderness ratio	$\bar{\lambda}_{LT,0} = \mathbf{0.4}$

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection	$\delta_{lim} = L_{s1} / 360 = \mathbf{5.6}$ mm
Maximum deflection span 1	$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{0.92}$ mm

PASS - Maximum deflection does not exceed deflection limit