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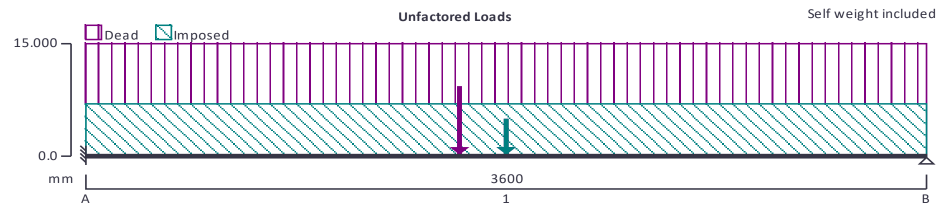
Project				Job no.	
BEAM 3 - 203x203x43kgUC S275				2023-7459	
Calcs for				Start page no./Revision	
Mr Colin Williams, 59 Oakhill Road, Horsham RH13 5LE				1	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
SB	12/10/2023	DB	12/10/2023	SB	12/10/2023

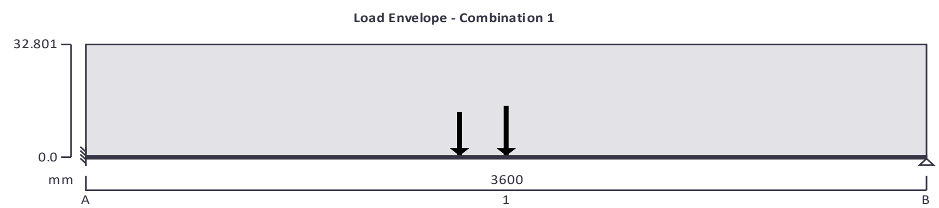
STEEL BEAM ANALYSIS & DESIGN (BS5950)

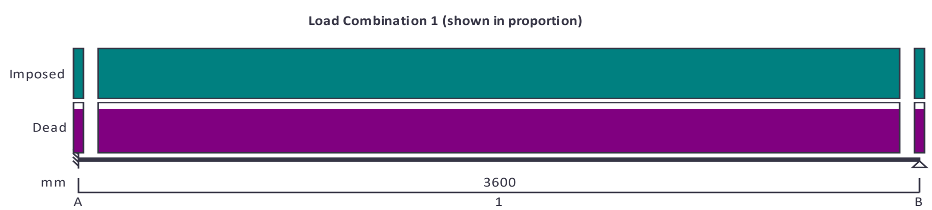
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

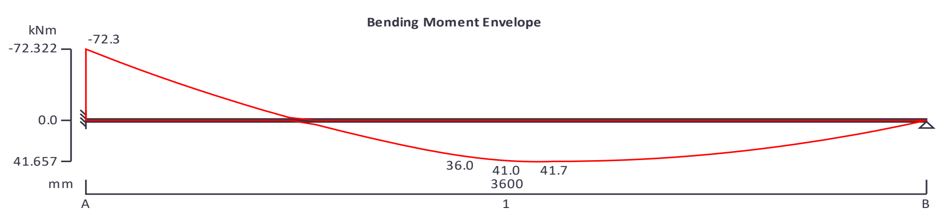
TEDDS calculation version 3.0.07

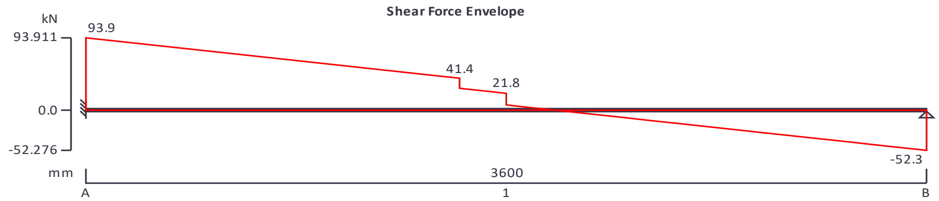
Minimum bearing
each end 150mm











Support conditions

Support A — Vertically restrained; Rotationally restrained

Support B — Vertically restrained; Rotationally free

Applied loading

Beam loads
- Dead self weight of beam × 1
- Dead full UDL 15 kN/m
- Imposed full UDL 6.98 kN/m
- Dead point load 5 kN at 1800 mm
- Imposed point load 5 kN at 1800 mm
- Dead point load 9.36 kN at 1600 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **41.7** kNm	M_{min} = **-72.3** kNm
Maximum shear	V_{max} = **93.9** kN	V_{min} = **-52.3** kN
Deflection	δ_{max} = **0.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **93.9** kN	R_{A_min} = **93.9** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **45.2** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **19.1** kN	
Maximum reaction at support B	R_{B_max} = **52.3** kN	R_{B_min} = **52.3** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **24.8** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **11** kN	

Section details

Section type — **UC 203x203x46 (BS4-1)**

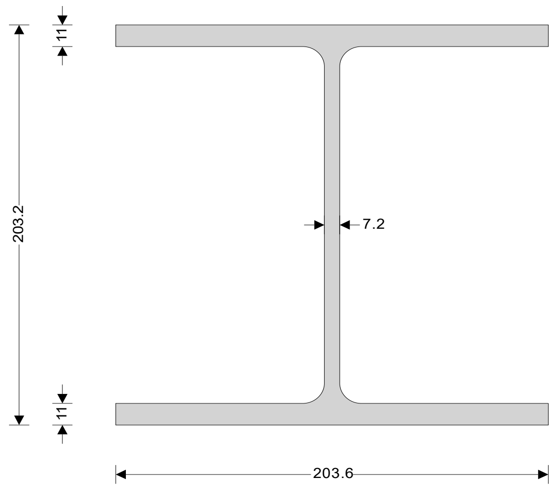
Steel grade — **S275**

From table 9: Design strength p_y

Thickness of element — max(T, t) = **11.0** mm

Design strength — p_y = **275** N/mm^2

Modulus of elasticity — E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis — K_x = **1.00**

Effective length factor in minor axis — K_y = **1.00**

Effective length factor for lateral-torsional buckling — $K_{LT.A}$ = **1.00**

$K_{LT.B}$ = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.00**

Internal compression parts - Table 11

Depth of section — d = **160.8** mm

$d / t = 22.3 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **101.8** mm

$b / T = 9.3 \times \varepsilon <= 10 \times \varepsilon$ — Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force — $F_v = max(abs(V_{max}), abs(V_{min}))$ = **93.9** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D$ = **1463** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v$ = **241.4** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **72.3** kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = min(p_y \times S_{xx}, 1.5 \times p_y \times Z_{xx})$ = **136.8** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.0 \times L_{s1}$ = **3600** mm

Slenderness ratio — $\lambda = L_E / r_{yy}$ = **70.117**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter	u = **0.847**
Torsional index	x = **17.713**
Slenderness factor	$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.865**
Ratio - cl.4.3.6.9	β_W = **1.000**
Equivalent slenderness - cl.4.3.6.7	$\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **51.361**
Limiting slenderness - Annex B.2.2	$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **34.310**

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant	α_{LT} = **7.0**
Perry factor	$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.119**
Euler stress	$p_E = \pi^2 \times E / \lambda_{LT}^2$ = **767** N/mm^2
	$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **566.8** N/mm^2
Bending strength - Annex B.2.1	$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **234.6** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment	M_2 = **1.1** kNm
Moment at centre-line of segment	M_3 = **41** kNm
Moment at three quarter point of segment	M_4 = **33.8** kNm
Maximum moment in segment	M_{abs} = **72.3** kNm
Maximum moment governing buckling resistance	$M_{LT} = M_{abs}$ = **72.3** kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44)$ = **0.555**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment	$M_b = p_b \times S_{xx}$ = **116.7** kNm
	M_b / m_{LT} = **210.1** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection	$\delta_{lim} = \min(14 \text{ mm}, L_{s1} / 360)$ = **10** mm
Maximum deflection span 1	$\delta = \max(abs(\delta_{max}), abs(\delta_{min}))$ = **0.909** mm

PASS - Maximum deflection does not exceed deflection limit