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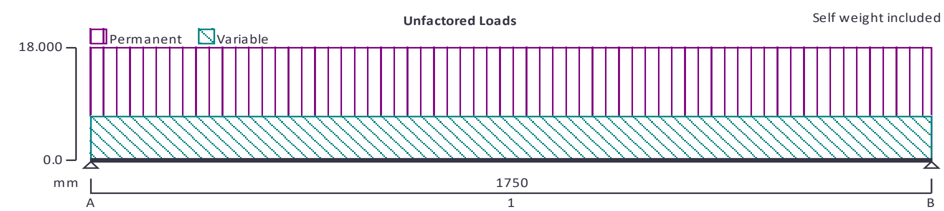
Project				Job no.	
Front Bay - Upper Structural Frame work				2023-7459	
Calcs for				Start page no./Revision	
Mr Shy Iqbal - 93 Bishopton Rd Stockton-on-Tees TS18 4PG				1	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
SB	16/10/2023	DB	16/10/2023	SB	16/10/2023

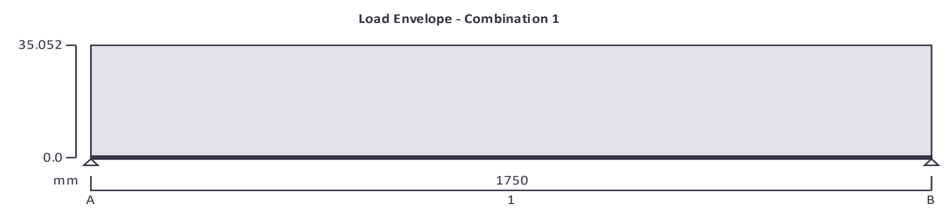
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

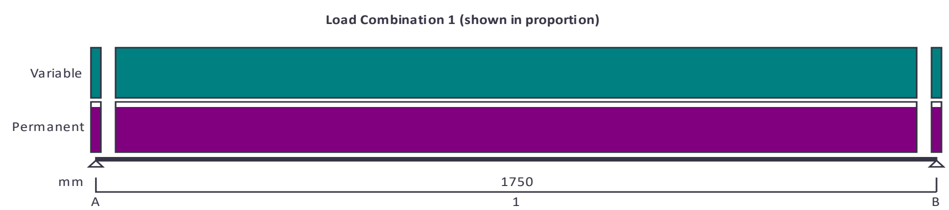
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

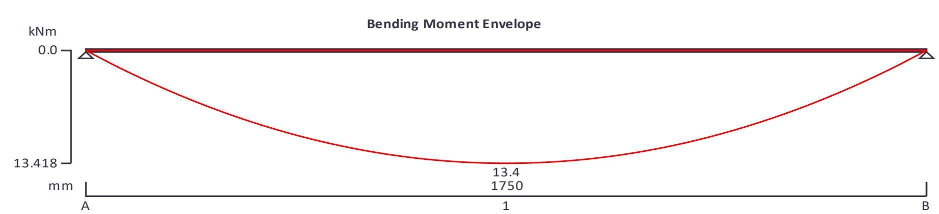
TEDDS calculation version 3.0.13

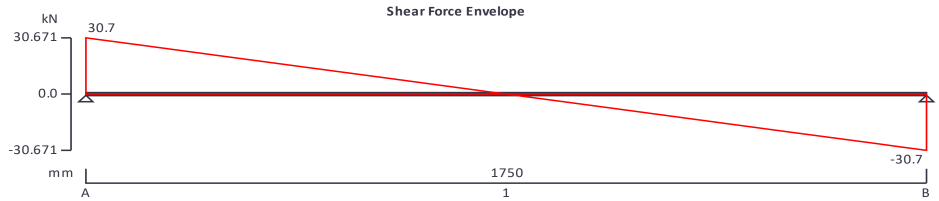
Calcs for Beam A, B, C - all 3 beams support loads and conditions are the same. Beam A 1750mm long, Beam B 1000mm long, Beam C 500mm long. The calculations are based the longest beam, Beam A. All Beams PASS.











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 18 kN/m
	Variable full UDL 7 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

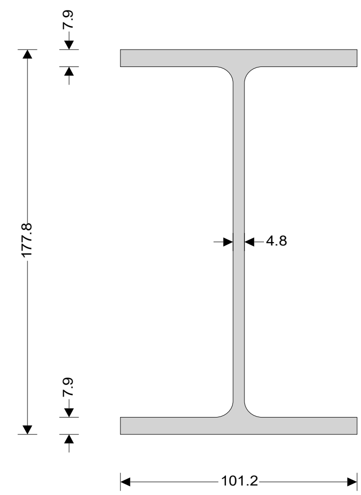
Maximum moment	M_{max} = **13.4** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **30.7** kN	V_{min} = **-30.7** kN
Deflection	δ_{max} = **0.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **30.7** kN	R_{A_min} = **30.7** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **15.9** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **6.1** kN	
Maximum reaction at support B	R_{B_max} = **30.7** kN	R_{B_min} = **30.7** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **15.9** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **6.1** kN	

Section details

Section type	**UB 178x102x19 (BS4-1)**
Steel grade	**S275**

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

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



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1.00**

Resistance of members to instability γ_{M1} = **1.00**

Resistance of tensile members to fracture γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

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 - Table 5.2 (sheet 1 of 3)

Width of section $c = d$ = **146.8** mm

$c / t_w = 33.1 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **40.6** mm

$c / t_f = 5.6 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **162** mm

Shear area factor η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **30.7** 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** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **156.4** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **13.4** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **47.1** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

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

Poissons ratio ν = **0.3**

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

Unrestrained length $L = 1.0 \times L_{s1}$ = **1750** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **116.2** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.637**

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 α_{LT} = **0.34**

Correction factor for rolled sections β = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.692**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.900**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.972**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.927**

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **43.6** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **4.9** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.3** mm

PASS - Maximum deflection does not exceed deflection limit

Check - capacity of the fillet welds connecting the beam web to the beam web – beams A, B ,C

Effective throat size of weld $a_{weld} = s_{weld} \times 0.7$ = **4.2** mm

Effective length of weld $l_{weld} = 2 \times (\min(l_{endplate}, 2 \times r_{supported} + d_{supported}) - 2 \times s_{weld})$ = **276.0** mm

Design strength of weld p_{weld} = **220** N/mm^2

Capacity of fillet welds $P_{weld} = p_{weld} \times l_{weld} \times a_{weld}$ = **255.0** kN

Utilisation factor $U_{check5weld} = Q / P_{weld}$ = **0.196**

Capacity of fillet weld : PASS