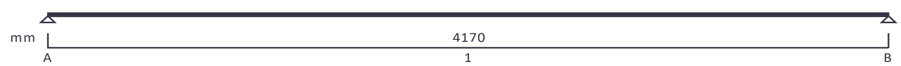
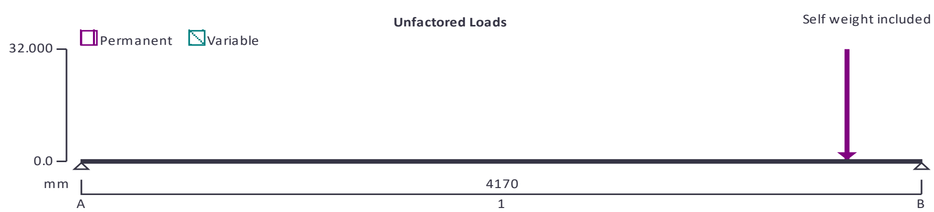
PlanningApplications.com Summer House, Upper Court Road Woldingham Surrey CR3 7BF support@planningapplications.com 07922 148 701	Project BEAM B - to support BEAM A - front bay roof area			Job no. 2023-7459	
	Calcs for Mr Ashley Mujer 12 Avondale Gardens Hounslow TW4 5HX			Start page no./Revision 1	
	Calcs by SB	Calcs date 28/09/2023	Checked by DB	Checked date 28/09/2023	Approved by SB	Approved date 28/09/2023

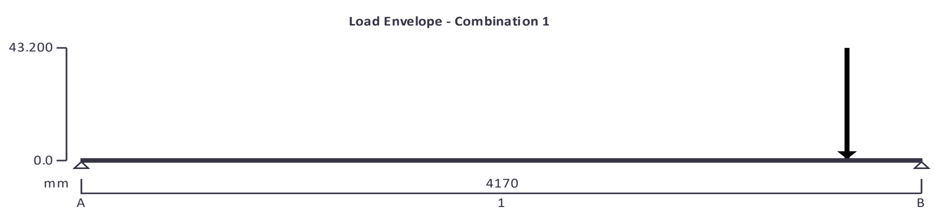
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

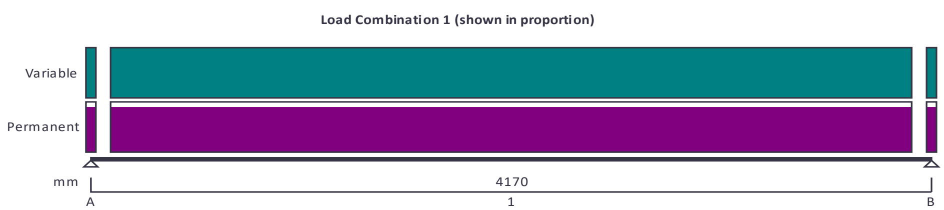
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

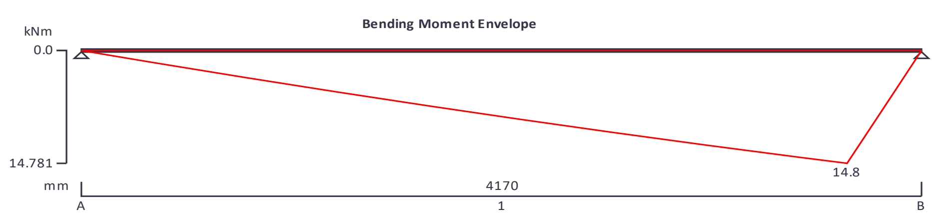
TEDDS calculation version 3.0.13

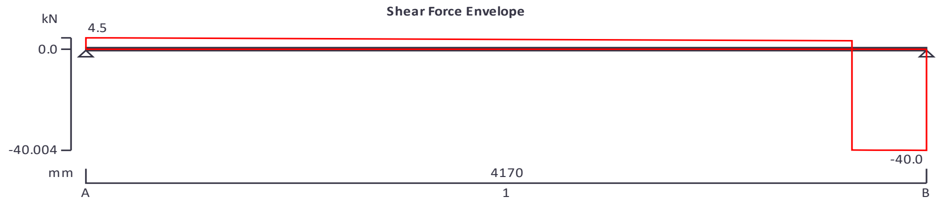












Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent point load 32 kN at 3800 mm

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

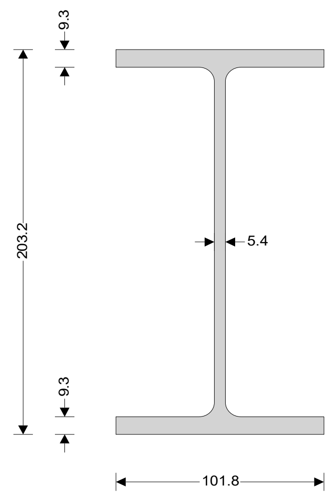
Maximum moment	M_{max} = **14.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **4.5** kN	V_{min} = **-40** kN
Deflection	δ_{max} = **0** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **4.5** kN	R_{A_min} = **4.5** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **3.3** kN	
Maximum reaction at support B	R_{B_max} = **40** kN	R_{B_min} = **40** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **29.6** kN	

Section details

Section type	**UB 203x102x23 (BS4-1)**
Steel grade	**S275**

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

Nominal thickness of element	$t = \max(t_f, t_w)$ = **9.3** 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 $c = d$ = **169.4** mm

$c / t_w = 33.9 \times \varepsilon <= 72 \times \varepsilon$ 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

$c / t_f = 4.7 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **184.6** mm

Shear area factor η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **40** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1238** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **196.6** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment	$M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **14.8** kNm
Design bending resistance moment - eq 6.13	$M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **64.4** 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	$g = \sqrt{[1 - (I_z / I_y)]}$ = **0.96**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2
Unrestrained length	$L = 1.0 \times L_{s1}$ = **4170** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **45.1** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.194**
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	$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.170**
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.582**
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.979**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.595**
Design buckling resistance moment - eq 6.55	$M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **38.3** 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	$\delta_{lim} = L_{s1} / 360$ = **11.6** mm
Maximum deflection span 1	$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0** mm

PASS - Maximum deflection does not exceed deflection limit