

	Project 46 Tredegar Road		Job No. 104643
	Part of Structure Cantilever terrace		Sheet No. 1
	Calculations by RL	Checked by SA	Date 28.10.21
CALCULATIONS			

The following calculations are for the cantilever terrace structure at 46 Tredegar Road

The calculations should be read in conjunction with drawings provided by HLN

All design accords with current edition the following:

BS EN 1991-1-1	Actions on Structures
BS EN 1993-1-1	Structural Steelwork
BS EN 1991-1-4	Wind actions
EN1996-1-1:2005 + A1:2012	Masonry
BS EN 1992-1-1 + UK NA (2004)	Concrete

Steelwork to have fire protection in accordance with the Building Regs

DESIGN FOR CONCRETE AND STEEL BASED ON:

TEKLA TEDDS

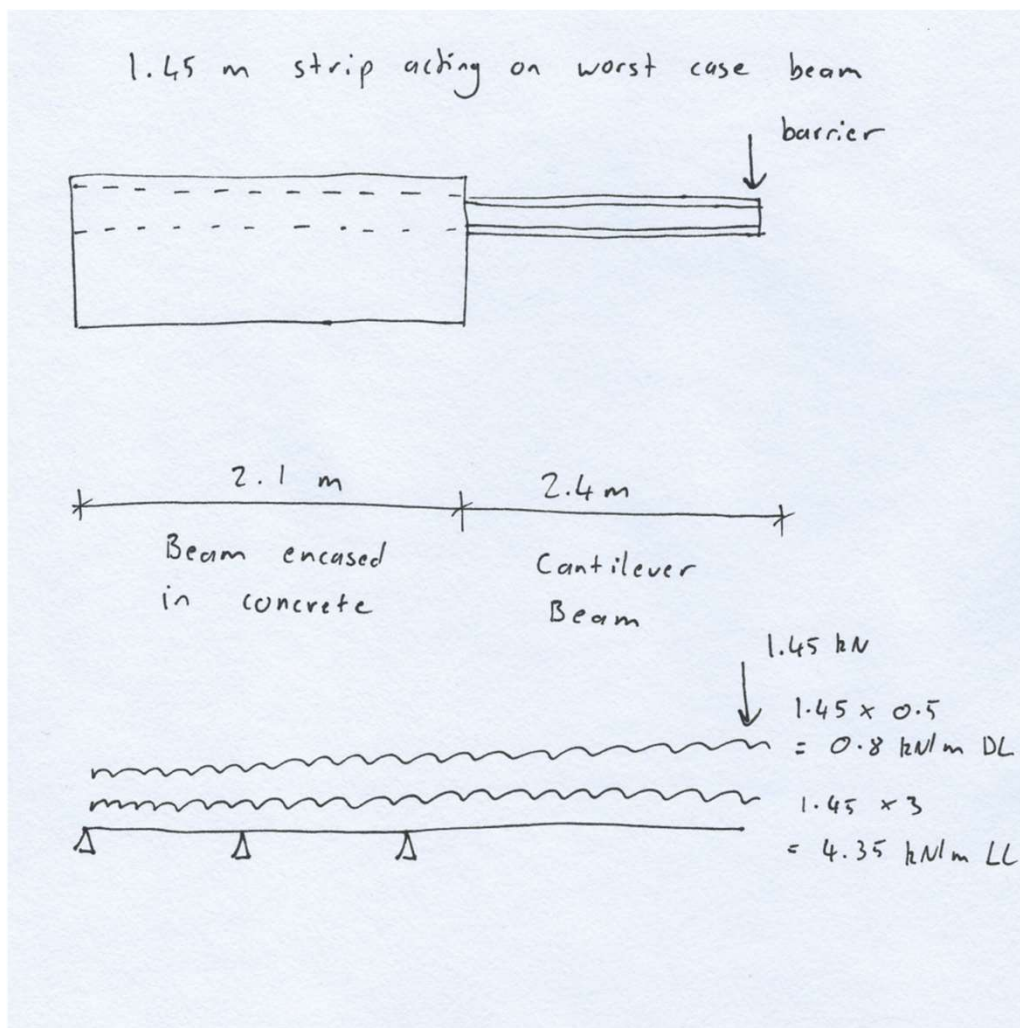
	Project 46 Tredegar Road		Job No. 104643
	Part of Structure Cantilever terrace		Sheet No. 2
	Calculations by RL	Checked by SA	Date 28.10.21
CALCULATIONS			

Typical Loadings (kN / m²)

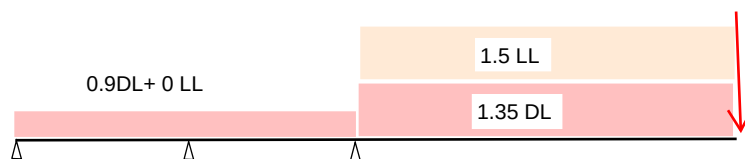
Deck

D.L.	Decking on 150x50mm joists @400c/c	0.50
	Total D.L.	<u>0.50</u> kN/m ²
L.L.	Residential Balcony 2.5 allow for 3	<u>3.00</u> kN/m ²
	Total SLS	<u>3.50</u> kN/m²

	Project 46 Tredegar Road	Job No. 104643
	Part of Structure Cantilever terrace	Sheet No. 3
	Calculations by RL	Checked by SA Date 28.10.21
CALCULATIONS		



Design case 1



Design case 2

Project 46 Tredegar				Job no. 104643	
Calcs for Primary Steel member - Design case 1				Start page no./Revision 1	
Calcs by RL	Calcs date 23/11/2021	Checked by SA	Checked date	Approved by	Approved date

STEEL MEMBER 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

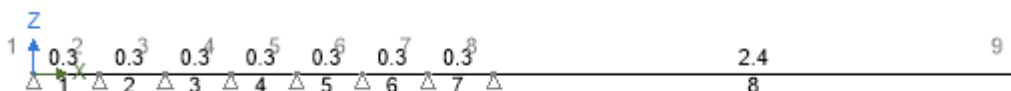
Tedds calculation version 4.4.06

ANALYSIS

Tedds calculation version 1.0.35

Geometry

Geometry (m) - Steel (EC3) - UB 203x133x25



Span	Length (m)	Section	Start Support	End Support
1	0.3	UB 203x133x25	Pinned	Pinned
2	0.3	UB 203x133x25	Pinned	Pinned
3	0.3	UB 203x133x25	Pinned	Pinned
4	0.3	UB 203x133x25	Pinned	Pinned
5	0.3	UB 203x133x25	Pinned	Pinned
6	0.3	UB 203x133x25	Pinned	Pinned
7	0.3	UB 203x133x25	Pinned	Pinned
8	2.4	UB 203x133x25	Pinned	Free

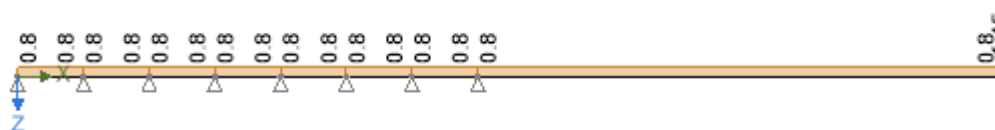
UB 203x133x25: Area 32 cm², Inertia Major 2340 cm⁴, Inertia Minor 308 cm⁴, Shear area parallel to Minor 12 cm², Shear area parallel to Major = 19 cm²

Steel (EC3): Density 7850 kg/m³, Youngs 210 kN/mm², Shear 80.8 kN/mm², Thermal 0.000012 °C⁻¹

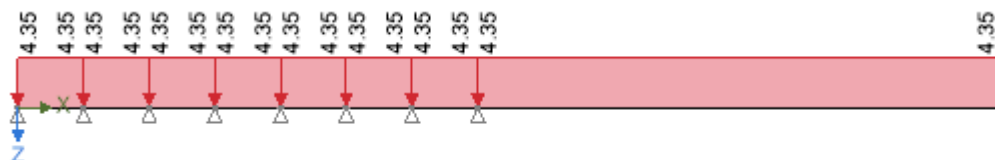
Loading

Self weight included

Permanent - Loading (kN/m,kN)



Imposed - Loading (kN/m)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50

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Load combination	Self Weight	Permanent	Imposed
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00

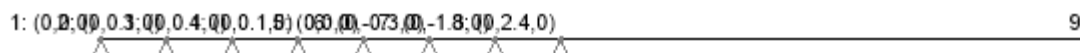
Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	UDL	GlobalZ	0.8 kN/m
Beam	Permanent	Point load	GlobalZ	1.45 kN at 4.5 m
Beam	Imposed	UDL	GlobalZ	4.35 kN/m

Results

Reactions

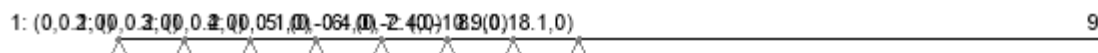
Self Weight - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



Load case: Self Weight

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0	0
2	0	0.1	0
3	0	0.1	0
4	0	0.1	0
5	0	0	0
6	0	-0.3	0
7	0	-1.3	0
8	0	2.4	0

Permanent - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



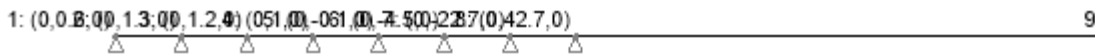
Load case: Permanent

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0.1	0
2	0	0.2	0
3	0	0.2	0
4	0	0.1	0
5	0	-0.4	0
6	0	-2.4	0
7	0	-10.9	0
8	0	18.1	0

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Calcs for Primary Steel member - Design case 1				Start page no./Revision 3	
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Imposed - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))

1: (0.0, 0.0, 1.3, 0.0, 1.2, 0.0) (0.5, 0.0, -0.6, 0.0, -7.5, 0.0) (2.7, 0.0, 42.7, 0.0)

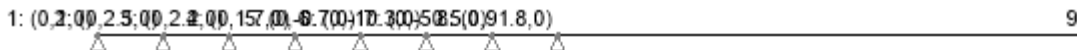


Load case: Imposed

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0.6	0
2	0	1.3	0
3	0	1.2	0
4	0	1	0
5	0	-0.1	0
6	0	-4.5	0
7	0	-22.7	0
8	0	42.7	0

1.35G + 1.5Q + 1.5RQ (Strength) - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))

1: (0.2, 0.0, 2.5, 0.0, 2.2, 0.0) (1.7, 0.0, -0.7, 0.0, -10.3, 0.0) (50.5, 0.0, 91.8, 0.0)

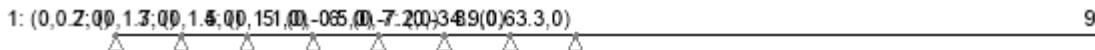


Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	1	0
2	0	2.5	0
3	0	2.2	0
4	0	1.7	0
5	0	-0.7	0
6	0	-10.3	0
7	0	-50.5	0
8	0	91.8	0

1.0G + 1.0Q + 1.0RQ (Service) - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))

1: (0.0, 0.2, 0.0, 1.3, 0.0, 1.5, 0.0) (1.5, 0.0, -0.5, 0.0, -7.2, 0.0) (3.4, 0.0, 63.3, 0.0)



Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0.7	0
2	0	1.7	0
3	0	1.5	0
4	0	1.1	0
5	0	-0.5	0
6	0	-7.2	0

Project 46 Tredegar				Job no. 104643	
Calcs for Primary Steel member - Design case 1				Start page no./Revision 4	
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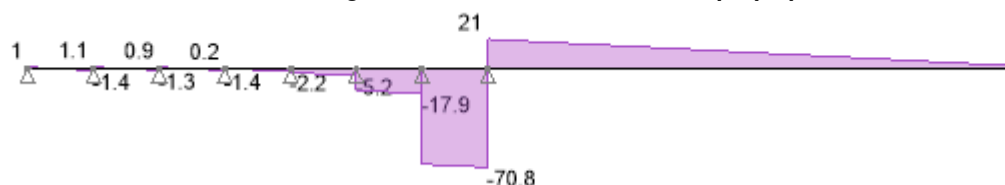
Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
7	0	-34.9	0
8	0	63.3	0

Forces

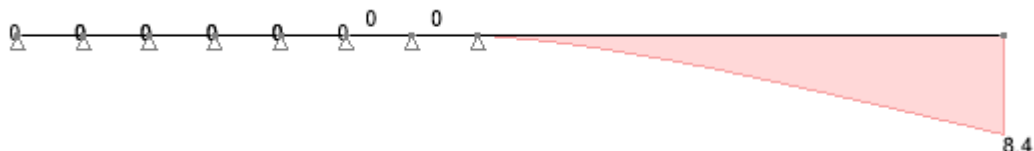
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Partial factors - Section 6.1

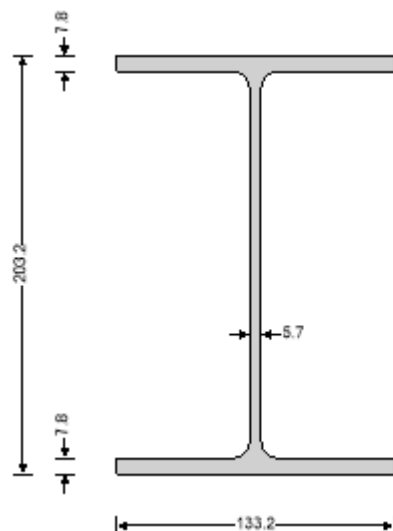
Resistance of cross-sections	$\gamma_{M0} = 1$
Resistance of members to instability	$\gamma_{M1} = 1$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.1$

Beam - Span 1 design

Section details

Section type	UB 203x133x25 (BS4-1)
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_r, t_w) = 7.8 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$

Project 46 Tredegar				Job no. 104643	
Calcs for Primary Steel member - Design case 1				Start page no./Revision 5	
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UB 203x133x25 (B54-1)
 Section depth, h, 203.2 mm
 Section breadth, b, 133.2 mm
 Mass of section, Mass, 25.1 kg/m
 Flange thickness, t_f, 7.8 mm
 Web thickness, t_w, 5.7 mm
 Root radius, r, 7.6 mm
 Area of section, A, 3197 mm²
 Radius of gyration about y-axis, i_y, 85.559 mm
 Radius of gyration about z-axis, i_z, 31.021 mm
 Elastic section modulus about y-axis, W_{el,y}, 230332 mm³
 Elastic section modulus about z-axis, W_{el,z}, 46190 mm³
 Plastic section modulus about y-axis, W_{pl,y}, 257731 mm³
 Plastic section modulus about z-axis, W_{pl,z}, 70944 mm³
 Second moment of area about y-axis, I_y, 23401694 mm⁴
 Second moment of area about z-axis, I_z, 3076268 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

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

$$c = d = 172.4 \text{ mm}$$

$$c / t_w = 30.2 = 32.7 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 56.1 \text{ mm}$$

$$c / t_f = 7.2 = 7.8 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check design at start of span

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm} \quad \eta = 1.000$$

$$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = 1 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1282 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 203.5 \text{ kN}$$

$$V_{y,Ed} / V_{c,y,Rd} = 0.005$$

PASS - Design shear resistance exceeds design shear force

Check design 129 mm along span

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 0.1 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 70.9 \text{ kNm}$$

$$M_{y,Ed} / M_{c,y,Rd} = 0.001$$

PASS - Design bending resistance moment exceeds design bending moment

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Calcs for Primary Steel member - Design case 1				Start page no./Revision 6	
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Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{0.914}$$

$$C_1 = 1 / k_c^2 = \mathbf{1.198}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

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

Unrestrained effective length

$$L = 1.0 \times L_{m1_s1_seg1_T} = \mathbf{300 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = \mathbf{8319.1 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{0.092}$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

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

Check design at end of span

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = \mathbf{187.6 \text{ mm}} \quad \eta = \mathbf{1.000}$$

$$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = \mathbf{1.4 \text{ kN}}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1282 \text{ mm}^2}$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{203.5 \text{ kN}}$$

$$V_{y,Ed} / V_{c,y,Rd} = \mathbf{0.007}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = \mathbf{0.1 \text{ kNm}}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{70.9 \text{ kNm}}$$

$$M_{y,Ed} / M_{c,y,Rd} = \mathbf{0.001}$$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{0.914}$$

$$C_1 = 1 / k_c^2 = \mathbf{1.198}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

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

Unrestrained effective length

$$L = 1.0 \times L_{m1_s1_seg1_B} = \mathbf{300 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = \mathbf{8319.1 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{0.092}$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

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

Beam - Span 3 design

Section details

Section type

$$\text{UB } 203 \times 133 \times 25 \text{ (BS4-1)}$$

Steel grade - EN 10025-2:2004

$$\text{S275}$$

Nominal thickness of element

$$t_{nom} = \max(t_f, t_w) = \mathbf{7.8 \text{ mm}}$$

Nominal yield strength

$$f_y = \mathbf{275 \text{ N/mm}^2}$$

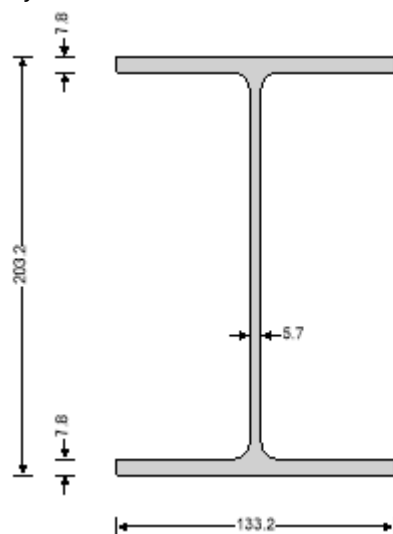
Nominal ultimate tensile strength

$$f_u = \mathbf{410 \text{ N/mm}^2}$$

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Modulus of elasticity

$E = 210000 \text{ N/mm}^2$



UB 203x133x25 (B54-1)
 Section depth, h, 203.2 mm
 Section breadth, b, 133.2 mm
 Mass of section, Mass, 25.1 kg/m
 Flange thickness, t_f , 7.8 mm
 Web thickness, t_w , 5.7 mm
 Root radius, r, 7.6 mm
 Area of section, A, 3197 mm²
 Radius of gyration about y-axis, I_y , 85.559 mm
 Radius of gyration about z-axis, I_z , 31.021 mm
 Elastic section modulus about y-axis, $W_{el,y}$, 230332 mm³
 Elastic section modulus about z-axis, $W_{el,z}$, 46190 mm³
 Plastic section modulus about y-axis, $W_{pl,y}$, 257731 mm³
 Plastic section modulus about z-axis, $W_{pl,z}$, 70944 mm³
 Second moment of area about y-axis, I_y , 23401694 mm⁴
 Second moment of area about z-axis, I_z , 3076268 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

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

$$c = d = 172.4 \text{ mm}$$

$$c / t_w = 30.2 = 32.7 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 56.1 \text{ mm}$$

$$c / t_f = 7.2 = 7.8 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check design at start of span

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm} \quad \eta = 1.000$$

$$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = 0.9 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1282 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 203.5 \text{ kN}$$

$$V_{y,Ed} / V_{c,y,Rd} = 0.005$$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 0.1 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 70.9 \text{ kNm}$$

$$M_{y,Ed} / M_{c,y,Rd} = 0.001$$

PASS - Design bending resistance moment exceeds design bending moment

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Calcs for Primary Steel member - Design case 1				Start page no./Revision 8	
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Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.574$$

$$C_1 = 1 / k_c^2 = 3.038$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained effective length

$$L = 1.0 \times L_{m1_s3_seg1_B} = 300 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 21102.2 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.058$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

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

Check design at end of span

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm} \quad \eta = 1.000$$

$$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = 1.4 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1282 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 203.5 \text{ kN}$$

$$V_{y,Ed} / V_{c,y,Rd} = 0.007$$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 0.2 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 70.9 \text{ kNm}$$

$$M_{y,Ed} / M_{c,y,Rd} = 0.002$$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.574$$

$$C_1 = 1 / k_c^2 = 3.038$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained effective length

$$L = 1.0 \times L_{m1_s3_seg1_B} = 300 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 21102.2 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.058$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

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

Beam - Span 8 design

Section details

Section type

$$UB \ 203 \times 133 \times 25 \text{ (BS4-1)}$$

Steel grade - EN 10025-2:2004

$$S275$$

Nominal thickness of element

$$t_{nom} = \max(t_f, t_w) = 7.8 \text{ mm}$$

Nominal yield strength

$$f_y = 275 \text{ N/mm}^2$$

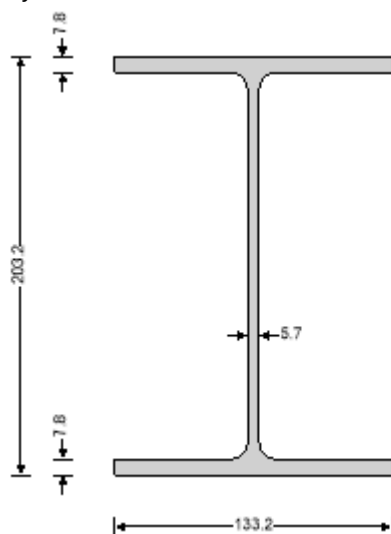
Nominal ultimate tensile strength

$$f_u = 410 \text{ N/mm}^2$$

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Modulus of elasticity

$E = 210000 \text{ N/mm}^2$



UB 203x133x25 (B54-1)
 Section depth, h , 203.2 mm
 Section breadth, b , 133.2 mm
 Mass of section, Mass, 25.1 kg/m
 Flange thickness, t_f , 7.8 mm
 Web thickness, t_w , 5.7 mm
 Root radius, r , 7.6 mm
 Area of section, A , 3197 mm²
 Radius of gyration about y-axis, i_y , 85.559 mm
 Radius of gyration about z-axis, i_z , 31.021 mm
 Elastic section modulus about y-axis, $W_{el,y}$, 230332 mm³
 Elastic section modulus about z-axis, $W_{el,z}$, 46190 mm³
 Plastic section modulus about y-axis, $W_{pl,y}$, 257731 mm³
 Plastic section modulus about z-axis, $W_{pl,z}$, 70944 mm³
 Second moment of area about y-axis, I_y , 23401694 mm⁴
 Second moment of area about z-axis, I_z , 3076268 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

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

$$c = d = 172.4 \text{ mm}$$

$$c / t_w = 30.2 = 32.7 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 56.1 \text{ mm}$$

$$c / t_f = 7.2 = 7.8 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check design at start of span

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm} \quad \eta = 1.000$$

$$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = 21 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1282 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 203.5 \text{ kN}$$

$$V_{y,Ed} / V_{c,y,Rd} = 0.103$$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 27.6 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 70.9 \text{ kNm}$$

$$M_{y,Ed} / M_{c,y,Rd} = 0.389$$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - For cantilever beams

$$k_c = 1$$

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Poissons ratio

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

$$\nu = 0.3$$

Shear modulus

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

Unrestrained effective length

$$L = 1.0 \times L_{m1_s8_seg1_B} = 2400 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 130.5 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.737$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

$$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0} - \text{Lateral torsional buckling cannot be ignored}$$

Check buckling resistance - 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

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.761$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.851$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 1.000$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = 0.851$$

Design buckling resistance moment - eq 6.55

$$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 60.3 \text{ kNm}$$

$$M_{y,Ed} / M_{b,y,Rd} = 0.457$$

PASS - Design buckling resistance moment exceeds design bending moment

Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)

Check design at end of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection

$$\delta_y = 8.4 \text{ mm}$$

Allowable deflection

$$\delta_{y,Allowable} = L_{m1_s8} / 180 = 13.3 \text{ mm}$$

$$\delta_y / \delta_{y,Allowable} = 0.629$$

PASS - Allowable deflection exceeds design deflection

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STEEL MEMBER 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

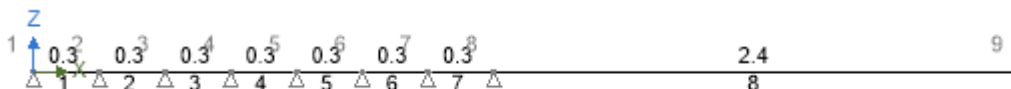
Tedds calculation version 4.4.06

ANALYSIS

Tedds calculation version 1.0.35

Geometry

Geometry (m) - Steel (EC3) - UB 203x133x25



Span	Length (m)	Section	Start Support	End Support
1	0.3	UB 203x133x25	Pinned	Pinned
2	0.3	UB 203x133x25	Pinned	Pinned
3	0.3	UB 203x133x25	Pinned	Pinned
4	0.3	UB 203x133x25	Pinned	Pinned
5	0.3	UB 203x133x25	Pinned	Pinned
6	0.3	UB 203x133x25	Pinned	Pinned
7	0.3	UB 203x133x25	Pinned	Pinned
8	2.4	UB 203x133x25	Pinned	Free

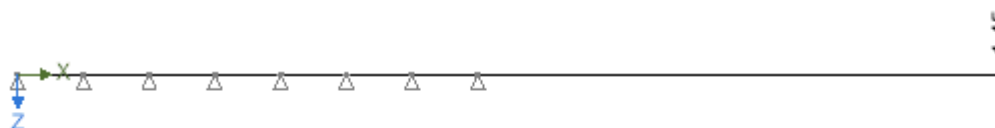
UB 203x133x25: Area 32 cm², Inertia Major 2340 cm⁴, Inertia Minor 308 cm⁴, Shear area parallel to Minor 12 cm², Shear area parallel to Major = 19 cm²

Steel (EC3): Density 7850 kg/m³, Youngs 210 kN/mm², Shear 80.8 kN/mm², Thermal 0.000012 °C⁻¹

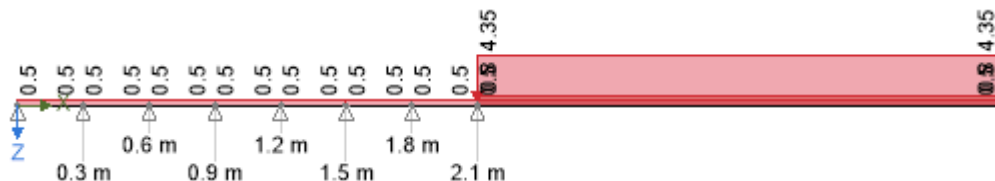
Loading

Self weight included

Permanent - Loading (kN)



Imposed - Loading (kN/m)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50

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Load combination	Self Weight	Permanent	Imposed
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	Point load	GlobalZ	1.45 kN at 4.5 m

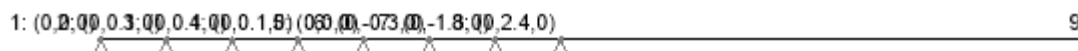
Element Loads

Element	Load case	Load Type	Orientation	Description
8	Imposed	UDL	GlobalZ	4.35 kN/m
8	Imposed	UDL	GlobalZ	0.8 kN/m
1	Imposed	UDL	GlobalZ	0.5 kN/m
2	Imposed	UDL	GlobalZ	0.5 kN/m
3	Imposed	UDL	GlobalZ	0.5 kN/m
4	Imposed	UDL	GlobalZ	0.5 kN/m
5	Imposed	UDL	GlobalZ	0.5 kN/m
6	Imposed	UDL	GlobalZ	0.5 kN/m
7	Imposed	UDL	GlobalZ	0.5 kN/m
8	Imposed	UDL	GlobalZ	0.5 kN/m

Results

Reactions

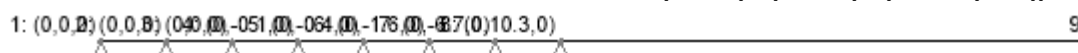
Self Weight - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



Load case: Self Weight

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0	0
2	0	0.1	0
3	0	0.1	0
4	0	0.1	0
5	0	0	0
6	0	-0.3	0
7	0	-1.3	0
8	0	2.4	0

Permanent - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))

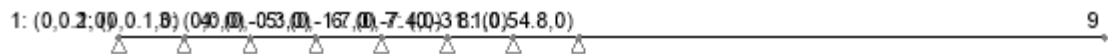


Load case: Permanent

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Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0	0
2	0	0	0
3	0	0	0
4	0	-0.1	0
5	0	-0.4	0
6	0	-1.6	0
7	0	-6.7	0
8	0	10.3	0

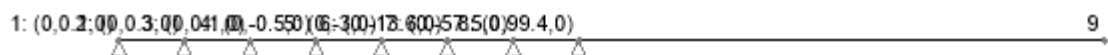
Imposed - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



Load case: Imposed

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0.1	0
2	0	0.1	0
3	0	0	0
4	0	-0.3	0
5	0	-1.7	0
6	0	-7.4	0
7	0	-31.1	0
8	0	54.8	0

1.35G + 1.5Q + 1.5RQ (Strength) - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))

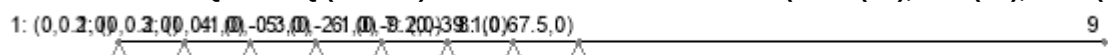


Load combination: 1.35G + 1.5Q + 1.5RQ (Strength)

Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0.1	0
2	0	0.3	0
3	0	0.1	0
4	0	-0.5	0
5	0	-3	0
6	0	-13.6	0
7	0	-57.5	0
8	0	99.4	0

forces used for holding down bolt design
self weight of concrete takes uplift forces

1.0G + 1.0Q + 1.0RQ (Service) - Local node reactions - Node: (Horiz (kN), Vert (kN), Mom (kNm))



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Load combination: 1.0G + 1.0Q + 1.0RQ (Service)

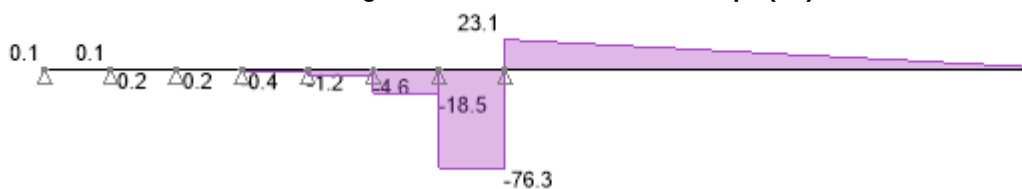
Node	Force		Moment My (kNm)
	Fx (kN)	Fz (kN)	
1	0	0.1	0
2	0	0.2	0
3	0	0.1	0
4	0	-0.3	0
5	0	-2.1	0
6	0	-9.2	0
7	0	-39.1	0
8	0	67.5	0

Forces

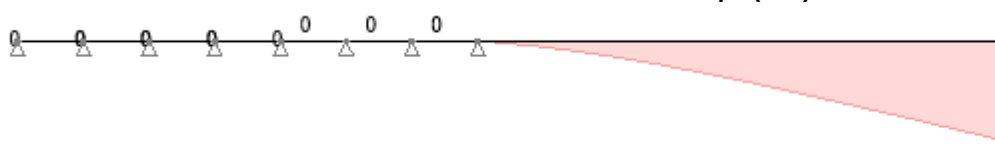
Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Service combinations - Deflection envelope (mm)



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1$
Resistance of members to instability	$\gamma_{M1} = 1$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.1$

Beam - Span 1 design

Section details

Section type	UB 203x133x25 (BS4-1)
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	$t_{nom} = \max(t_r, t_w) = 7.8 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$

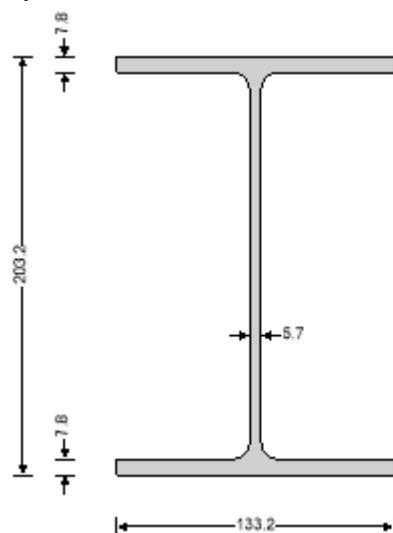
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Nominal ultimate tensile strength

$f_u = 410 \text{ N/mm}^2$

Modulus of elasticity

$E = 210000 \text{ N/mm}^2$



UB 203x133x25 (B34-1)
 Section depth, h, 203.2 mm
 Section breadth, b, 133.2 mm
 Mass of section, Mass, 25.1 kg/m
 Flange thickness, t_f , 7.8 mm
 Web thickness, t_w , 5.7 mm
 Root radius, r, 7.6 mm
 Area of section, A, 3197 mm²
 Radius of gyration about y-axis, i_y , 85.559 mm
 Radius of gyration about z-axis, i_z , 31.021 mm
 Elastic section modulus about y-axis, $W_{el,y}$, 230332 mm³
 Elastic section modulus about z-axis, $W_{el,z}$, 46190 mm³
 Plastic section modulus about y-axis, $W_{pl,y}$, 257731 mm³
 Plastic section modulus about z-axis, $W_{pl,z}$, 70944 mm³
 Second moment of area about y-axis, I_y , 23401694 mm⁴
 Second moment of area about z-axis, I_z , 3076268 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

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

$$c = d = 172.4 \text{ mm}$$

$$c / t_w = 30.2 = 32.7 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 56.1 \text{ mm}$$

$$c / t_f = 7.2 = 7.8 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check design at start of span

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm} \quad \eta = 1.000$$

$$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = 0.1 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1282 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 203.5 \text{ kN}$$

$$V_{y,Ed} / V_{c,y,Rd} = 0.001$$

PASS - Design shear resistance exceeds design shear force

Check design 114 mm along span

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 0 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 70.9 \text{ kNm}$$

$$M_{y,Ed} / M_{c,y,Rd} = 0$$

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PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.691$$

$$C_1 = 1 / k_c^2 = 2.095$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained effective length

$$L = 1.0 \times L_{m1_s1_seg1_T} = 300 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 14552.9 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.07$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

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

Check design at end of span

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm} \quad \eta = 1.000$$

$$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = 0.2 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1282 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 203.5 \text{ kN}$$

$$V_{y,Ed} / V_{c,y,Rd} = 0.001$$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 0 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 70.9 \text{ kNm}$$

$$M_{y,Ed} / M_{c,y,Rd} = 0$$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.691$$

$$C_1 = 1 / k_c^2 = 2.095$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained effective length

$$L = 1.0 \times L_{m1_s1_seg1_B} = 300 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 14552.9 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.07$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

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

Beam - Span 3 design

Section details

Section type

$$UB \ 203 \times 133 \times 25 \text{ (BS4-1)}$$

Steel grade - EN 10025-2:2004

$$S275$$

Nominal thickness of element

$$t_{nom} = \max(t_f, t_w) = 7.8 \text{ mm}$$

Nominal yield strength

$$f_y = 275 \text{ N/mm}^2$$

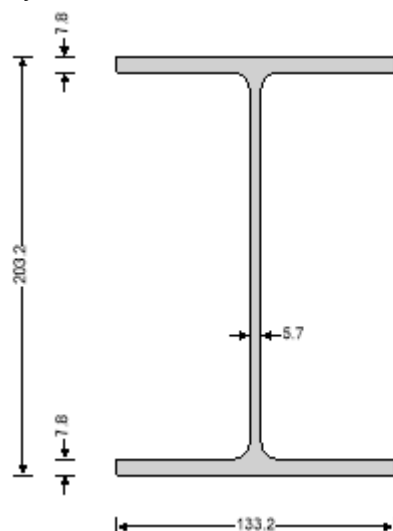
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Nominal ultimate tensile strength

$f_u = 410 \text{ N/mm}^2$

Modulus of elasticity

$E = 210000 \text{ N/mm}^2$



UB 203x133x25 (B34-1)
 Section depth, h, 203.2 mm
 Section breadth, b, 133.2 mm
 Mass of section, Mass, 25.1 kg/m
 Flange thickness, t_f , 7.8 mm
 Web thickness, t_w , 5.7 mm
 Root radius, r, 7.6 mm
 Area of section, A, 3197 mm²
 Radius of gyration about y-axis, i_y , 85.559 mm
 Radius of gyration about z-axis, i_z , 31.021 mm
 Elastic section modulus about y-axis, $W_{el,y}$, 230332 mm³
 Elastic section modulus about z-axis, $W_{el,z}$, 46190 mm³
 Plastic section modulus about y-axis, $W_{pl,y}$, 257731 mm³
 Plastic section modulus about z-axis, $W_{pl,z}$, 70944 mm³
 Second moment of area about y-axis, I_y , 23401694 mm⁴
 Second moment of area about z-axis, I_z , 3076268 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

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

$$c = d = 172.4 \text{ mm}$$

$$c / t_w = 30.2 = 32.7 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 56.1 \text{ mm}$$

$$c / t_f = 7.2 = 7.8 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check design at start of span

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm} \quad \eta = 1.000$$

$$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = 0.1 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1282 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 203.5 \text{ kN}$$

$$V_{y,Ed} / V_{c,y,Rd} = 0$$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 0 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 70.9 \text{ kNm}$$

$$M_{y,Ed} / M_{c,y,Rd} = 0$$

PASS - Design bending resistance moment exceeds design bending moment

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Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.727$$

$$C_1 = 1 / k_c^2 = 1.893$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained effective length

$$L = 1.0 \times L_{m1_s3_seg1_B} = 300 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 13152.3 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.073$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

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

Check design at end of span

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm} \quad \eta = 1.000$$

$$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = 0.4 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1282 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 203.5 \text{ kN}$$

$$V_{y,Ed} / V_{c,y,Rd} = 0.002$$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 0.1 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 70.9 \text{ kNm}$$

$$M_{y,Ed} / M_{c,y,Rd} = 0.002$$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.727$$

$$C_1 = 1 / k_c^2 = 1.893$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained effective length

$$L = 1.0 \times L_{m1_s3_seg1_B} = 300 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 13152.3 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.073$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

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

Beam - Span 8 design

Section details

Section type

$$UB 203 \times 133 \times 25 \text{ (BS4-1)}$$

Steel grade - EN 10025-2:2004

$$S275$$

Nominal thickness of element

$$t_{nom} = \max(t_f, t_w) = 7.8 \text{ mm}$$

Nominal yield strength

$$f_y = 275 \text{ N/mm}^2$$

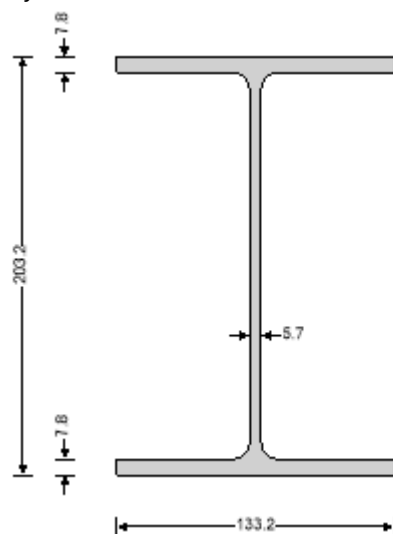
Nominal ultimate tensile strength

$$f_u = 410 \text{ N/mm}^2$$

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Modulus of elasticity

$E = 210000 \text{ N/mm}^2$



UB 203x133x25 (B54-1)
 Section depth, h, 203.2 mm
 Section breadth, b, 133.2 mm
 Mass of section, Mass, 25.1 kg/m
 Flange thickness, t_f , 7.8 mm
 Web thickness, t_w , 5.7 mm
 Root radius, r, 7.6 mm
 Area of section, A, 3197 mm²
 Radius of gyration about y-axis, i_y , 85.559 mm
 Radius of gyration about z-axis, i_z , 31.021 mm
 Elastic section modulus about y-axis, $W_{el,y}$, 230332 mm³
 Elastic section modulus about z-axis, $W_{el,z}$, 46190 mm³
 Plastic section modulus about y-axis, $W_{pl,y}$, 257731 mm³
 Plastic section modulus about z-axis, $W_{pl,z}$, 70944 mm³
 Second moment of area about y-axis, I_y , 23401694 mm⁴
 Second moment of area about z-axis, I_z , 3076268 mm⁴

Lateral restraint

Both flanges have lateral restraint at supports only

Consider Combination 1 - 1.35G + 1.5Q + 1.5RQ (Strength)

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

$$c = d = 172.4 \text{ mm}$$

$$c / t_w = 30.2 = 32.7 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 56.1 \text{ mm}$$

$$c / t_f = 7.2 = 7.8 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check design at start of span

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm} \quad \eta = 1.000$$

$$h_w / t_w = 32.9 = 35.6 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = 23.1 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1282 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 203.5 \text{ kN}$$

$$V_{y,Ed} / V_{c,y,Rd} = 0.113$$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 30.1 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 70.9 \text{ kNm}$$

$$M_{y,Ed} / M_{c,y,Rd} = 0.424$$

PASS - Design bending resistance moment exceeds design bending moment

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Calcs for Primary Steel member - design case 2				Start page no./Revision 10	
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Slenderness ratio for lateral torsional buckling

Correction factor - For cantilever beams $k_c = 1$
 $C_1 = 1 / k_c^2 = 1$
Poissons ratio $\nu = 0.3$
Shear modulus $G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained effective length $L = 1.0 \times L_{m1_s8_seg1_B} = 2400 \text{ mm}$
Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = 130.5 \text{ kNm}$
Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.737$
Limiting slenderness ratio $\bar{\lambda}_{LT,0} = 0.4$
 $\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Check buckling resistance - 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 $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.761$
LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.851$
Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 1.000$
Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = 0.851$
Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 60.3 \text{ kNm}$
 $M_{y,Ed} / M_{b,y,Rd} = 0.499$
PASS - Design buckling resistance moment exceeds design bending moment

Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)

Check design at end of span

Check y-y axis deflection - Section 7.2.1

Maximum deflection $\delta_y = 9 \text{ mm}$
Allowable deflection $\delta_{y,Allowable} = L_{m1_s8} / 180 = 13.3 \text{ mm}$
 $\delta_y / \delta_{y,Allowable} = 0.675$
PASS - Allowable deflection exceeds design deflection

Project 46 Tredegar Road				Job no. 104643	
Calcs for Holding down bolts				Start page no./Revision 1	
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PULL-OUT CAPACITY OF HOLDING DOWN BOLTS

TEDDS calculation version 1.0.01

These calculations assume that there is no coexistent shear in the holding-down bolts.

Tension capacity of bolts

Maximum tension force per bolt

$F_t = 58.0$ kN

Try M16 (Grade 8.8) bolts

$P_t = 70.7$ kN $d_{nut} = 13$ mm

Check $F_t \leq P_t$ Pass - Bolt tension capacity

Pull-out resistance of concrete

Overall length of bolts

$L_{bolt} = 300$ mm

Clear projection of bolt above nut

$L_{proj} = 25$ mm

Thickness of washer, baseplate & grout

$t_{was} = 10$ mm $t_p = 10$ mm $t_{gr} = 25$ mm

Depth of embedment

$d_e = L_{bolt} - L_{proj} - d_{nut} - t_{was} - t_p - t_{gr} = 217$ mm

From BCSA HDBolts document, effective conical surface area for two bolts $A_{ec} = 3779.4$ cm²

Depth of embedment selected

$d_e = 217$ mm

Design spacing of bolts (c/c)

$X_{ccs} = 300$ mm

Shear strength of concrete

$v_c = 0.34$ N/mm²

Design pull-out resistance, per bolt

$P_{tc} = A_{ec} \times v_c / 2 = 64.3$ kN

Check $F_t \leq P_{tc}$ Pass - Concrete pull-out resistance

Bolts require adequate anchorage by suitably sized washer plates or equivalent under the bolt heads.

BS 5950: Part 1: 2000: Sections 4.13 & 6.6

'Holding down systems for steel stanchions' BCSA / Constrado 1980

Project 46 Tredegar St				Job no. 105643	
Calcs for External Joists				Start page no./Revision 1	
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TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.04

Joist details

Description 50 x 150 C24 timber joists

Joist spacing $S_{Joist} = 400 \text{ mm}$



Forces input on Joist

Vertical permanent load on joist $F_{G_Joist} = 1.00 \text{ kN/m}^2$

Vertical imposed load on joist $F_{Q_Joist} = 4.00 \text{ kN/m}^2$

Vertical snow load on joist $F_{S_Joist} = 1.00 \text{ kN/m}^2$

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G_Joist} \times S_{Joist} = 0.40 \text{ kN/m}$

Vertical imposed load on joist $p_Q = F_{Q_Joist} \times S_{Joist} = 1.60 \text{ kN/m}$

Vertical snow load on joist $p_S = F_{S_Joist} \times S_{Joist} = 0.40 \text{ kN/m}$

ANALYSIS

Tedds calculation version 1.0.35

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.35G + 1.50Q + $\psi_s 1.50S$ (Strength)	1.35	1.50	0.75	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00
1.00G + 1.00Q + $\psi_s 1.00S$ (Service)	1.00	1.00	0.50	0.00

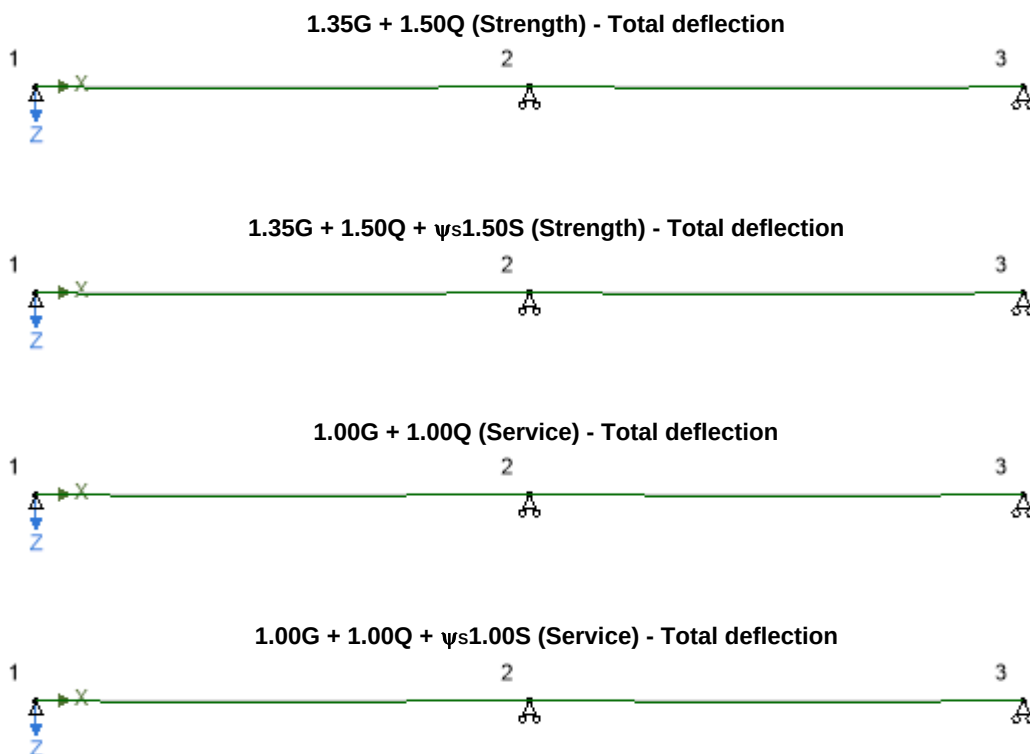
Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.4 kN/m at 0 m to 2.83 m
Member	Imposed	UDL	GlobalZ	1.6 kN/m at 0 m to 2.83 m
Member	Snow	UDL	GlobalZ	0.4 kN/m at 0 m to 2.83 m

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Results

Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.07516	
2	0	0	0	
3	0	0	-0.07516	

Load combination: 1.35G + 1.50Q + $\psi_s 1.50S$ (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.08272	
2	0	0	0	
3	0	0	-0.08272	

Load combination: 1.00G + 1.00Q (Service)

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Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.05119	
2	0	0	0	
3	0	0	-0.05119	

Load combination: 1.00G + 1.00Q + ψ s1.00S (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.05624	
2	0	0	0	
3	0	0	-0.05624	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	8.4
1.35G + 1.50Q + ψ s1.50S (Strength)	0	9.3
1.00G + 1.00Q (Service)	0	5.7
1.00G + 1.00Q + ψ s1.00S (Service)	0	6.3

Element end forces
Load combination: 1.35G + 1.50Q (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.415	1	0	-1.6	0
		2	0	-2.6	-0.7
2	1.415	2	0	-2.6	0.7
		3	0	-1.6	0

Load combination: 1.35G + 1.50Q + ψ s1.50S (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.415	1	0	-1.8	0
		2	0	-2.9	-0.8
2	1.415	2	0	-2.9	0.8
		3	0	-1.8	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.415	1	0	-1.1	0
		2	0	-1.8	-0.5
2	1.415	2	0	-1.8	0.5
		3	0	-1.1	0

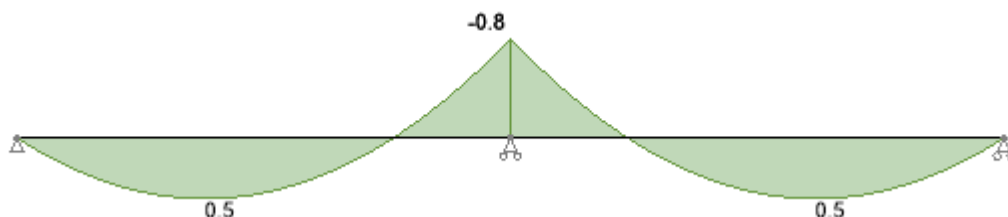
Project 46 Tredegar St				Job no. 105643	
Calcs for External Joists				Start page no./Revision 4	
Calcs by RL	Calcs date 11/11/2021	Checked by	Checked date	Approved by	Approved date

Load combination: 1.00G + 1.00Q + ψ s1.00S (Service)

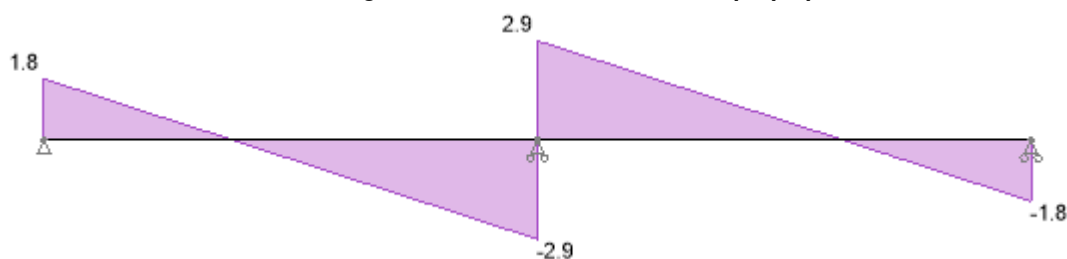
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	1.415	1	0	-1.2	0
		2	0	-2	-0.5
2	1.415	2	0	-2	0.5
		3	0	-1.2	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0.54	0		0.5 (max)	0.4
	1.415	2.9 (max abs)	-2.9	-0.7	-0.8 (min)

Tedds calculation version 2.2.09

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 $\gamma_M = 1.300$

Member details

Load duration - cl.2.3.1.2 Medium-term

Service class - cl.2.3.1.3 2

Timber section details

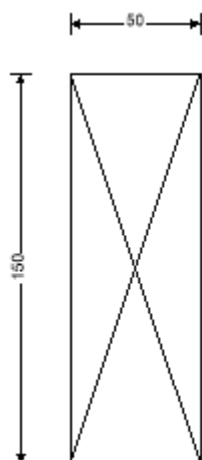
Number of timber sections in member N = 1

Breadth of sections b = 50 mm

Depth of sections h = 150 mm

Timber strength class - EN 338:2016 Table 1 C24

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50x150 timber section

Cross-sectional area, A , 7500 mm²
 Section modulus, W_y , 187500 mm³
 Section modulus, W_z , 62500 mm³
 Second moment of area, I_y , 14062500 mm⁴
 Second moment of area, I_z , 1562500 mm⁴
 Radius of gyration, r_y , 43.3 mm
 Radius of gyration, r_z , 14.4 mm
Timber strength class C24
 Characteristic bending strength, $f_{t,k}$, 24 N/mm²
 Characteristic shear strength, $f_{s,k}$, 4 N/mm²
 Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm²
 Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm²
 Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm²
 Mean modulus of elasticity, $E_{0.95,mean}$, 11000 N/mm²
 Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm²
 Shear modulus of elasticity, G_{mean} , 690 N/mm²
 Characteristic density, ρ_k , 350 kg/m³
 Mean density, ρ_{mean} , 420 kg/m³

Span details

Bearing length $L_b = 100$ mm

Consider Combination 2 - 1.35G + 1.50Q + ψ_s 1.50S (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.8$
 Deformation factor - Table 3.2 $k_{def} = 0.8$
 Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 0.7$
 Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$
 System strength factor - cl.6.6 $k_{sys} = 1.1$

Check design at end of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 5.745$ kN
 Effective contact length $L_{b,ef} = L_b = 100$ mm
 Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 1.149$ N/mm²
 Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.692$ N/mm²
 $\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.679$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 2.873$ kN
 Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.858$ N/mm²
 Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.708$ N/mm²
 $\tau_{y,d} / f_{v,y,d} = 0.317$

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 0.779$ kNm
 Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 4.157$ N/mm²
 Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 16.246$ N/mm²
 $\sigma_{m,y,d} / f_{m,y,d} = 0.256$

PASS - Design bending strength exceeds design bending stress

Project 46 Tredegar St				Job no. 105643	
Calcs for External Joists				Start page no./Revision 6	
Calcs by RL	Calcs date 11/11/2021	Checked by	Checked date	Approved by	Approved date

Consider Combination 4 - 1.00G + 1.00Q + ψ s1.00S (Service)

Check design 627 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$$\delta_y = 0.5 \text{ mm}$$

Quasi-permanent variable load factor

$$\psi_2 = 0.3$$

Final deflection with creep

$$\delta_{y, \text{Final}} = \delta_y \times (1 + k_{\text{def}}) = 0.8 \text{ mm}$$

Allowable deflection

$$\delta_{y, \text{Allowable}} = L_{m1_s1} / 250 = 5.7 \text{ mm}$$

$$\delta_{y, \text{Final}} / \delta_{y, \text{Allowable}} = 0.147$$

PASS - Allowable deflection exceeds final deflection