Calculation Sheet

PROJECT TITLE Carpenters Cottage-Uxbridge		PROJECT No 001107	
TITLE Chimney Stack and wall support		DATE 11/03/2022	SHEET No 01
BY E	CHECKED BY	SHEET REF	REV

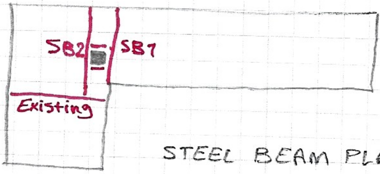
Reference | Calculations | Output

The purpose of following calculation is to design the Steel beam support for gable wall and chimney Stack.

LOADING:

chimney Stack wall or gable wall

$$S.W = 21 \frac{kN}{m^3} \times 0.1 = 2.1 \frac{kN}{m^2}$$



STEEL BEAM PLAN

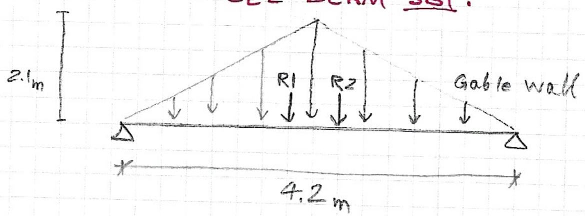
SB1 beam supports gable wall / SB2 trim chimney Stack

Calculation Sheet

PROJECT TITLE Carpenters Cottage - Uxbridge		PROJECT No 001107	
TITLE Chimney Stack and wall support		DATE 11/03/2022	SHEET No. 02
BY E	CHECKED BY	SHEET REF	REV

Reference | Calculations | Output

DESIGN OF STEEL BEAM SB1:



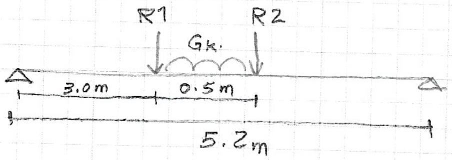
$G_k = 2.1 \frac{kN}{m^2} \times 2.1_m = 4.41 \frac{kN}{m}$

$R_1 = R_2 = 1.6\ kN$ (See below)

See Tedds calcs page 3

use UB 203x102x23

DESIGN OF STEEL BEAM SB2:



$R_1 = R_2 = 2.1 \frac{kN}{m2} \times 3.0_m \times \frac{0.5m}{2} = 1.6\ kN$

$G_K = 2.1 \frac{kN}{m2} \times 3.0_m = 6.3 \frac{kN}{m}$

see Tedds calcs page

USE UB 203x133x25

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

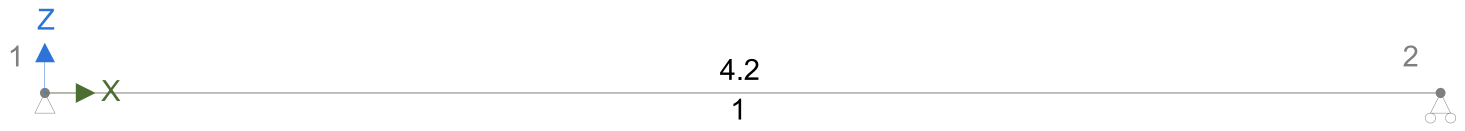
Tedds calculation version 4.4.00

ANALYSIS

Tedds calculation version 1.0.28

Geometry

Geometry (m) - Steel (EC3) - UB 203x102x23

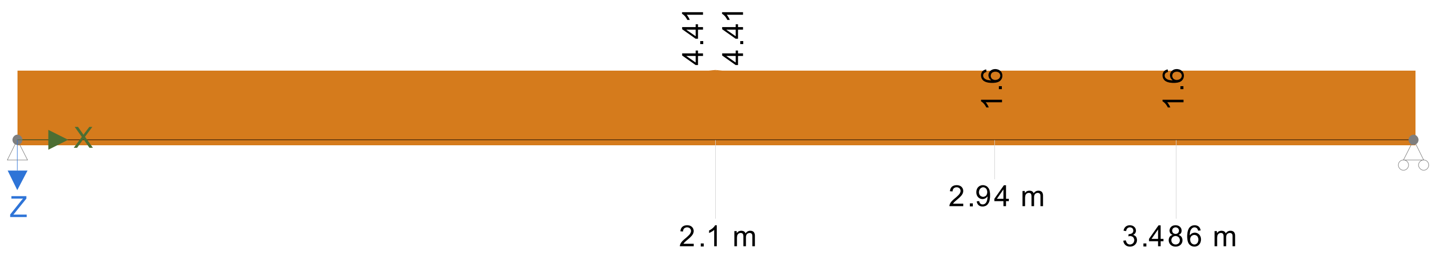


Span	Length (m)	Section	Start Support	End Support
1	4.2	UB 203x102x23	Pinned	Roller Pin X
UB 203x102x23: Area 29 cm^2, Inertia Major 2105 cm^4, Inertia Minor 164 cm^4, Shear area parallel to Minor 11 cm^2, Shear area parallel to Major = 17 cm^2				
Steel (EC3): Density 7850 kg/m^3, Youngs 210 kN/mm^2, Shear 80.8 kN/mm^2, Thermal 0.000012 °C^{-1}				

Loading

Self weight included

Permanent - Loading (kN/m,kN)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
1.0G + 1.0Q + 1.0RQ (Service)	1.00	1.00	1.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Beam	Permanent	VDL	GlobalZ	0 kN/m at 0 m to 4.41 kN/m at 2.1 m
Beam	Permanent	VDL	GlobalZ	4.41 kN/m at 2.1 m to 0 kN/m at 4.2 m
Beam	Permanent	Point load	GlobalZ	1.6 kN at 2.94 m
Beam	Permanent	Point load	GlobalZ	1.6 kN at 3.486 m

Tekla Tedds	Project Carpenters Cottage - Uxbridge				Job no. 001107	
	Calcs for Steel beam SB1				Start page no./Revision 4	
	Calcs by E	Calcs date 12/03/2022	Checked by	Checked date	Approved by	Approved date

Results

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 γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of tensile members to fracture γ_{M2} = **1.1**

Beam design

Section details

Section type UB 203x102x23 (BS4-1)

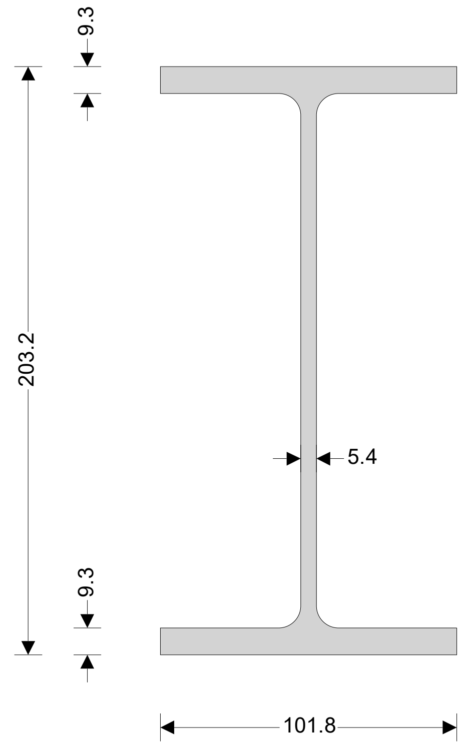
Steel grade - EN 10025-2:2004 S275

Nominal thickness of element t_{nom} = 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



UB 203x102x23 (BS4-1)
Section depth, h, 203.2 mm
Section breadth, b, 101.8 mm
Mass of section, Mass, 23.1 kg/m
Flange thickness, t_f, 9.3 mm
Web thickness, t_w, 5.4 mm
Root radius, r, 7.6 mm
Area of section, A, 2940 mm^2
Radius of gyration about y-axis, i_y, 84.615 mm
Radius of gyration about z-axis, i_z, 23.609 mm
Elastic section modulus about y-axis, $W_{el.y}$, 207175 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 32194 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 234069 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 49753 mm^3
Second moment of area about y-axis, I_y, 21048934 mm^4
Second moment of area about z-axis, I_z, 1638698 mm^4

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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \mathbf{0.92}$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = d = \mathbf{169.4}$ mm

$c / t_w = 31.4 = 33.9 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{40.6}$ mm

$c / t_f = 4.4 = 4.7 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check design at start of span

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{184.6}$ mm — $\eta = \mathbf{1.000}$

$h_w / t_w = 34.2 = 37 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed} = \mathbf{7.9}$ kN

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

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

$V_{y,Ed} / V_{c,y,Rd} = \mathbf{0.04}$

PASS - Design shear resistance exceeds design shear force

Check design 2269 mm along span

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed} = \mathbf{11.6}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{64.4}$ kNm

$M_{y,Ed} / M_{c,y,Rd} = \mathbf{0.181}$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

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

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

Poissons ratio ν = **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)]$ = **80769** N/mm²

Unrestrained effective length $L = 1.0 \times L_{m1\_s1\_seg1\_T}$ = **4200** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **42.9** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.224**

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 α_{LT} = **0.34**

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

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.202**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.565**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.981**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.576**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **37.1** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.314**

PASS - Design buckling resistance moment exceeds design bending moment

<u>Check design at end of span</u>

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **184.6** mm η = **1.000**

$h_w / t_w = 34.2 = 37 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed}$ = **10.2** 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** mm²

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

$V_{y,Ed} / V_{c,y,Rd}$ = **0.052**

PASS - Design shear resistance exceeds design shear force

<u>Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)</u>

<u>Check design 2150 mm along span</u>

Check y-y axis deflection - Section 7.2.1

Maximum deflection δ_y = **3.6** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 360$ = **11.7** mm

$\delta_y / \delta_{y,Allowable}$ = **0.309**

PASS - Allowable deflection exceeds design deflection

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

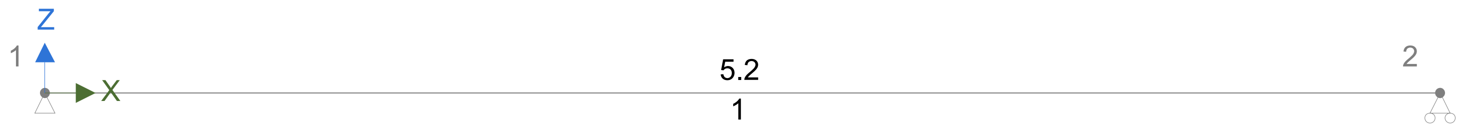
Tedds calculation version 4.4.00

ANALYSIS

Tedds calculation version 1.0.28

Geometry

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

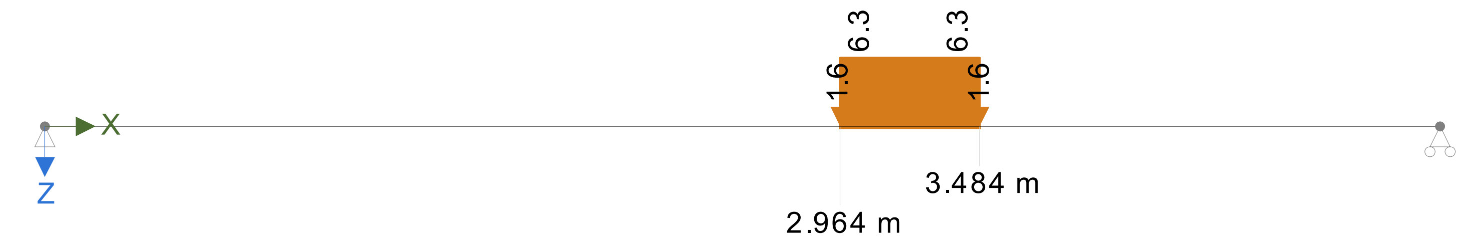


Span	Length (m)	Section	Start Support	End Support
1	5.2	UKB 203x133x25	Pinned	Roller Pin X
UKB 203x133x25: Area 32 cm^2, Inertia Major 2340 cm^4, Inertia Minor 308 cm^4, Shear area parallel to Minor 12 cm^2, Shear area parallel to Major = 19 cm^2				
Steel (EC3): Density 7850 kg/m^3, Youngs 210 kN/mm^2, Shear 80.8 kN/mm^2, Thermal 0.000012 °C^{-1}				

Loading

Self weight included

Permanent - Loading (kN/m,kN)



Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
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	6.3 kN/m at 2.964 m to 3.484 m
Beam	Permanent	Point load	GlobalZ	1.6 kN at 2.964 m
Beam	Permanent	Point load	GlobalZ	1.6 kN at 3.484 m

Results

Forces

Strength combinations - Moment envelope (kNm)

Strength combinations - Shear envelope (kN)

Service combinations - Deflection envelope (mm)

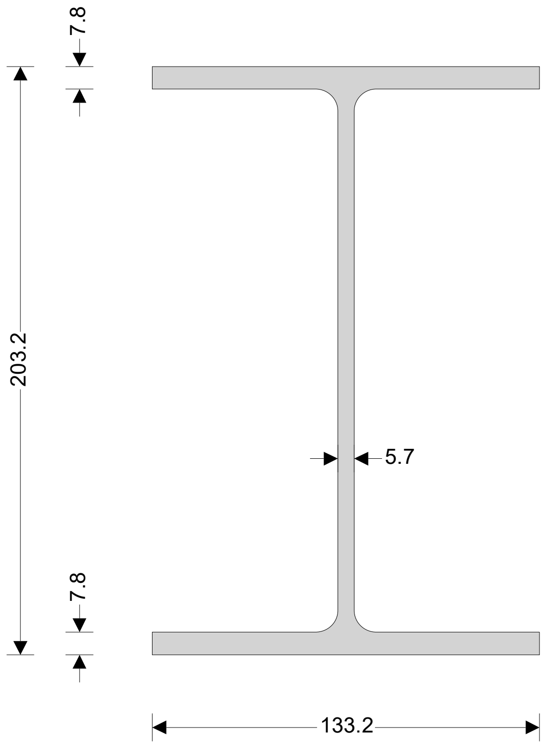
Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1**
Resistance of members to instability	γ_{M1} = **1**
Resistance of tensile members to fracture	γ_{M2} = **1.1**

Beam design

Section details

Section type	UKB 203x133x25 (Tata Steel Advance)
Steel grade - EN 10025-2:2004	S275
Nominal thickness of element	t_{nom} = max(t_f, t_w) = **7.8** 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



UKB 203x133x25 (Tata Steel Advance)
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 mm^2
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 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 46190 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 257731 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 70944 mm^3
Second moment of area about y-axis, I_y, 23401694 mm^4
Second moment of area about z-axis, I_z, 3076268 mm^4

Lateral restraint

Both flanges have lateral restraint at supports only

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \mathbf{0.92}$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = d = \mathbf{172.4}$ mm

$c / t_w = 30.2 = 32.7 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{56.1}$ mm

$c / t_f = 7.2 = 7.8 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

<u>**Check design at start of span**</u>

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{187.6}$ mm — $\eta = \mathbf{1.000}$

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

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed} = \mathbf{4.2}$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1282}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} = \mathbf{203.5}$ kN

$V_{y,Ed} / V_{c,y,Rd} = \mathbf{0.021}$

PASS - Design shear resistance exceeds design shear force

<u>**Check design 3082 mm along span**</u>

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed} = \mathbf{11}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{70.9}$ kNm

$M_{y,Ed} / M_{c,y,Rd} = \mathbf{0.155}$

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — $k_c = \mathbf{0.94}$

$C_1 = 1 / k_c^2 = \mathbf{1.132}$

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

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

Unrestrained effective length $L = 1.0 \times L_{m1\_s1\_seg1\_T} = \mathbf{5200}$ mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))} = \mathbf{46.2}$ kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{1.239}$

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

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - 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 $\alpha_{LT} = \mathbf{0.34}$

Correction factor for rolled sections $\beta = \mathbf{0.75}$

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.218}$

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.557}$

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.982}$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.568}$

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{40.2}$ kNm

$M_{y,Ed} / M_{b,y,Rd} = \mathbf{0.274}$

PASS - Design buckling resistance moment exceeds design bending moment

<u>Check design at end of span</u>

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = \mathbf{187.6}$ mm $\eta = \mathbf{1.000}$

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

Shear buckling resistance can be ignored

Design shear force $V_{y,Ed} = \mathbf{6.3}$ kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1282}$ mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,y,Rd} = V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0} = \mathbf{203.5}$ kN

$V_{y,Ed} / V_{c,y,Rd} = \mathbf{0.031}$

PASS - Design shear resistance exceeds design shear force

<u>Consider Combination 2 - 1.0G + 1.0Q + 1.0RQ (Service)</u>

<u>Check design 2767 mm along span</u>

Check y-y axis deflection - Section 7.2.1

Maximum deflection $\delta_y = \mathbf{4.1}$ mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 360 = \mathbf{14.4}$ mm

$\delta_y / \delta_{y,Allowable} = \mathbf{0.284}$

PASS - Allowable deflection exceeds design deflection

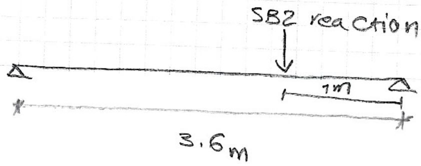
Calculation Sheet

PROJECT TITLE Carpenters cottage - Uxbridge		PROJECT No 001107	
TITLE Chimney stack support		DATE 11/03/2022	SHEET No. 11
BY E	CHECKED BY	SHEET REF	REV .

Reference | Calculations | Output

ASSESSMENT OF RECENTLEY INSTALLED BEAM:

There's a new steel UC installed on site that we will utilise to support SB2.



SB2 reaction (R) = $\frac{6.3\ kN}{1.35} = 4.7\ KN$

Section size UC 152x152x23

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

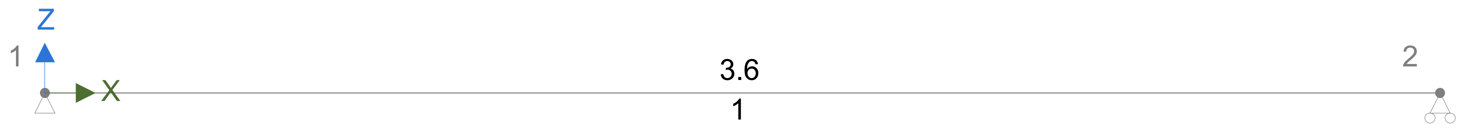
Tedds calculation version 4.4.00

ANALYSIS

Tedds calculation version 1.0.28

Geometry

Geometry (m) - Steel (EC3) - UC 152x152x23



Span	Length (m)	Section	Start Support	End Support
1	3.6	UC 152x152x23	Pinned	Roller Pin X
UC 152x152x23: Area 29 cm^2, Inertia Major 1250 cm^4, Inertia Minor 400 cm^4, Shear area parallel to Minor 9 cm^2, Shear area parallel to Major = 19 cm^2				
Steel (EC3): Density 7850 kg/m^3, Youngs 210 kN/mm^2, Shear 80.8 kN/mm^2, Thermal 0.000012 $°C^{-1}$				

Loading

Self weight included

Permanent - Loading (kN)

Load combination factors

Load combination	Self Weight	Permanent	Imposed
1.35G + 1.5Q + 1.5RQ (Strength)	1.35	1.35	1.50
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	4.7 kN at 0.972 m

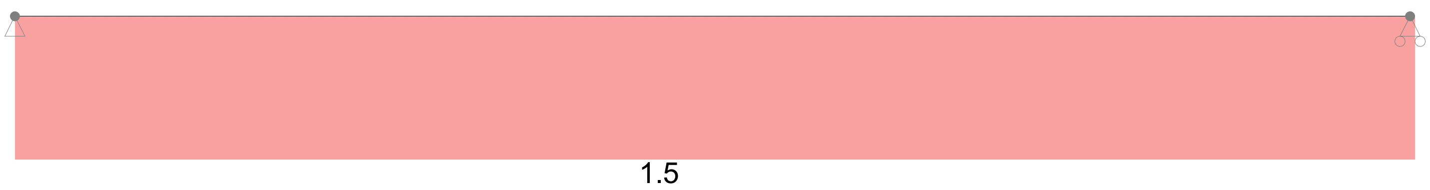
Results

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 γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of tensile members to fracture γ_{M2} = **1.1**

Beam design

Section details

Section type UC 152x152x23 (BS4-1)

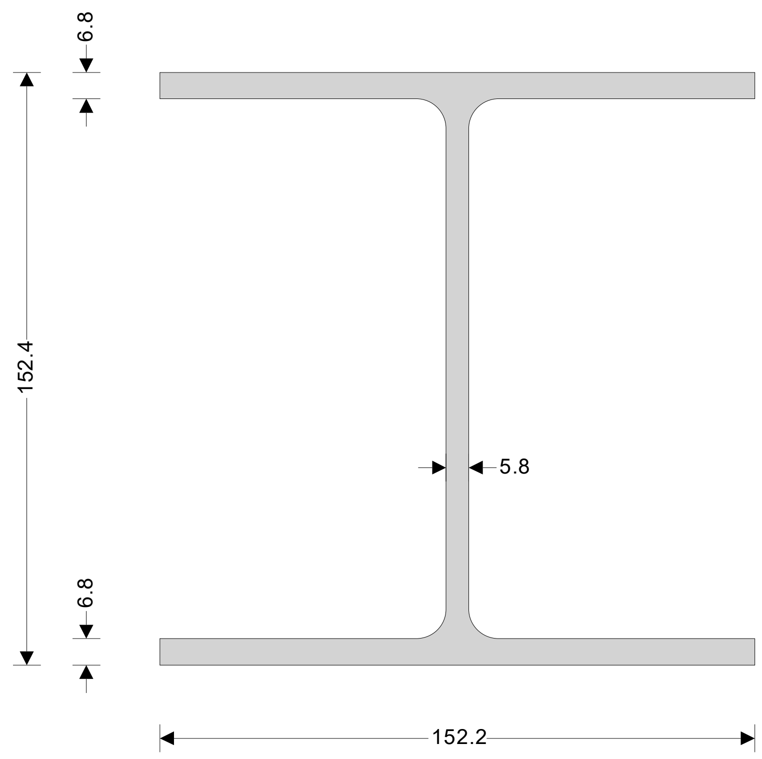
Steel grade - EN 10025-2:2004 S275

Nominal thickness of element t_{nom} = max(t_f, t_w) = **6.8** 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



UC 152x152x23 (BS4-1)
Section depth, h, 152.4 mm
Section breadth, b, 152.2 mm
Mass of section, Mass, 23 kg/m
Flange thickness, t_f, 6.8 mm
Web thickness, t_w, 5.8 mm
Root radius, r, 7.6 mm
Area of section, A, 2925 mm^2
Radius of gyration about y-axis, i_y, 65.372 mm
Radius of gyration about z-axis, i_z, 36.979 mm
Elastic section modulus about y-axis, $W_{el.y}$, 164016 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 52552 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 181982 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 80156 mm^3
Second moment of area about y-axis, I_y, 12498039 mm^4
Second moment of area about z-axis, I_z, 3999186 mm^4

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 — $c = d$ = **123.6** mm

$c / t_w = 21.3 = 23.1 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **65.6** mm

$c / t_f = 9.6 = 10.4 \times \varepsilon <= 14 \times \varepsilon$ Class 3

Section is class 3

<u>Check design at start of span</u>

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **138.8** mm η = **1.000**

$h_w / t_w = 23.9 = 25.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed}$ = **5.2** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **997** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **158.4** kN

$V_{y,Ed} / V_{pl,y,Rd}$ = **0.033**

PASS - Design shear resistance exceeds design shear force

<u>Check design 972 mm along span</u>

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **138.8** mm η = **1.000**

$h_w / t_w = 23.9 = 25.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{y,Ed}$ = **4.9** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **997** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **158.4** kN

$V_{y,Ed} / V_{pl,y,Rd}$ = **0.031**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment — $M_{y,Ed}$ = **4.9** kNm

Design bending resistance moment - eq 6.14 — $M_{c,y,Rd} = M_{el,y,Rd} = W_{el.y} \times f_y / \gamma_{M0}$ = **45.1** kNm

$M_{y,Ed} / M_{c,y,Rd}$ = **0.108**

PASS - Design bending resistance moment exceeds design bending moment

Slenderness ratio for lateral torsional buckling

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

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

Poissons ratio — ν = **0.3**

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

Unrestrained effective length — $L = 1.0 \times L_{m1\_s1\_seg1\_T}$ = **3600** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / L^2 \times \sqrt{(I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z))}$ = **76.4** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{el.y} \times f_y / M_{cr})}$ = **0.768**

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 α_{LT} = **0.34**

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

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.784**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.834**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.970**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.860**

Design buckling resistance moment - eq 6.55 $M_{b,y,Rd} = \chi_{LT,mod} \times W_{el.y} \times f_y / \gamma_{M1}$ = **38.8** kNm

$M_{y,Ed} / M_{b,y,Rd}$ = **0.126**

PASS - Design buckling resistance moment exceeds design bending moment

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

Check design 1608 mm along span

Check y-y axis deflection - Section 7.2.1

Maximum deflection δ_y = **1.5** mm

Allowable deflection $\delta_{y,Allowable} = L_{m1\_s1} / 360$ = **10** mm

$\delta_y / \delta_{y,Allowable}$ = **0.152**

PASS - Allowable deflection exceeds design deflection