CALCULATION SHEET

JOB TITLE:	JOB NUMBER / FILE:	CALCULATION NUMBER:		Form
24 HEATH DRIVE	162637			
CALCULATION:	CALCULATION BY:	DATE:	CHECKED BY:	
G-12 LOADING	HD	04/12/18		

CALCULATIONS:

REF | OUTPUT

LOADING G-12

FIRST FLOOR

TIMBER FLOOR. 2m SPAN.

DL 2m x 1.02 kN/m² = 2.04 kN/m

LL 2m x 1.5 kN/m² = 3 kN/m

FIRST FLOOR STUD WALL -ASSUMED

WALL LOAD "

0.55 kN/m² x 3.2m = 1.76 kN/m

GROUND FLOOR

TIMBER FLOOR SPAN 2.5m

DL 2.5m x 1.02 kN/m² = 2.65 kN/m

LL 2.5m x 1.5 kN/m² = 3.75 kN/m

WALL LOAD

DL = 2.54 kN/m² x 3.3m = 8.4 kN/m

TOTAL LOAD:

DL = 14.9 kN/m + 8.2

LL = 6.8 kN/m

3/12/19

CCM GF 250THK RC SLAB

GF LOADING

RC SLAB 2/1 x 7.8 = 7.8 kN/m DL (94)

1 x 1.5 = 1.5 kN/m LL

TIMBER 1 x 1.05 = 1.05 kN/m DL

1 x 1.5 = 1.5 kN/m LL

8.85 DL + 3 LL kN/m

SEE TEDDS

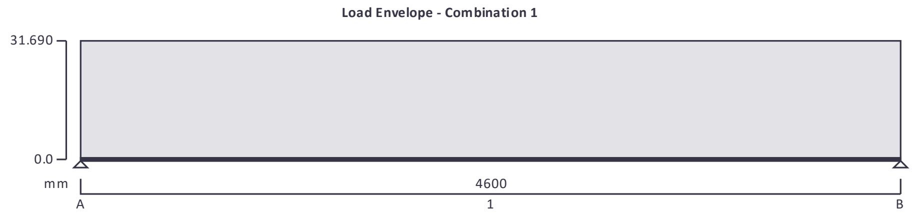
→ 203UC52 ↑ ⇒ 203UC71.

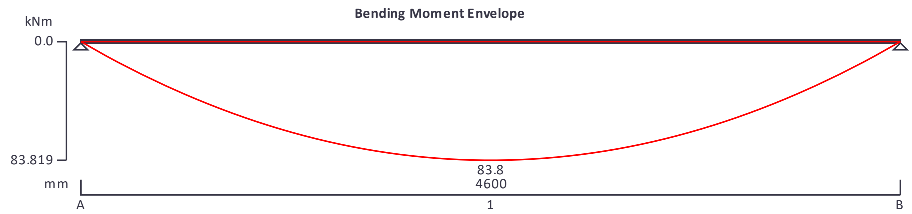
Form Structural Design 77 St John Street London EC1M 4NN	Project 24 HEATH DRIVE				Job no. 162637	
	Calcs for BEAM G-12				Start page no./Revision 1	
	Calcs by HD	Calcs date 15/01/2019	Checked by ROM	Checked date	Approved by	Approved date

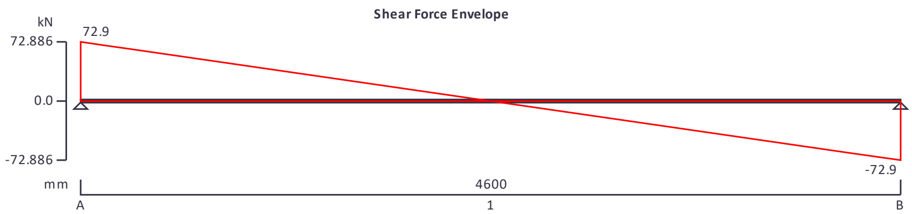
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 full UDL 15 kN/m	
	Imposed full UDL 7 kN/m	

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Imposed × 1.50
		Permanent × 1.35
		Imposed × 1.50
	Support B	Permanent × 1.35
		Imposed × 1.50

Analysis results

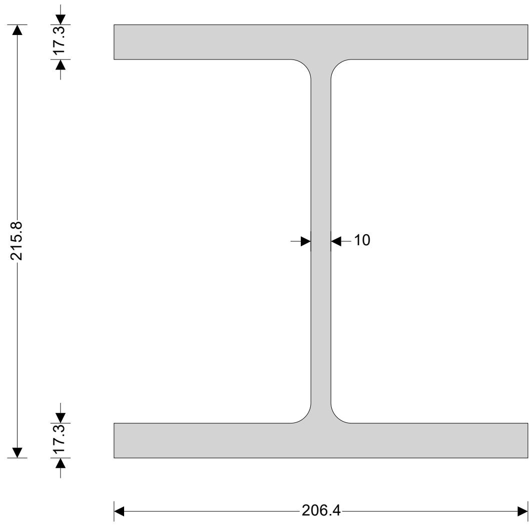
Maximum moment	M_{max} = **83.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **72.9** kN	V_{min} = **-72.9** kN
Deflection	δ_{max} = **8.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **72.9** kN	R_{A_min} = **72.9** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **36.1** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **16.1** kN	
Maximum reaction at support B	R_{B_max} = **72.9** kN	R_{B_min} = **72.9** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **36.1** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **16.1** kN	

Section details

Section type	**UC 203x203x71 (BS4-1)**
Steel grade	**S355**

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

Nominal thickness of element	$t = max(t_f, t_w)$ = **17.3** mm
Nominal yield strength	f_y = **345** N/mm^2
Nominal ultimate tensile strength	f_u = **470** 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

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = \mathbf{0.83}$

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

Width of section — $c = d = \mathbf{160.8}$ mm

$c / t_w = 19.5 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{88}$ mm

$c / t_f = 6.2 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{181.2}$ mm

Shear area factor — $\eta = \mathbf{1.000}$

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{72.9}$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{2427}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{483.5}$ kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{83.8}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{275.6}$ kNm

Slenderness ratio for lateral torsional buckling

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

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

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.817}$

Poissons ratio — $\nu = \mathbf{0.3}$

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

Unrestrained length — $L = 1.0 \times L_{s1} = \mathbf{4600}$ mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{652.8}$ kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{0.65}$

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

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

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

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{0.701}$

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.894}$

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.971}$

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.920}$

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{253.7}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection δ_{lim} = min(10 mm, L_{s1} / 250) = **10** mm

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **8.271** mm

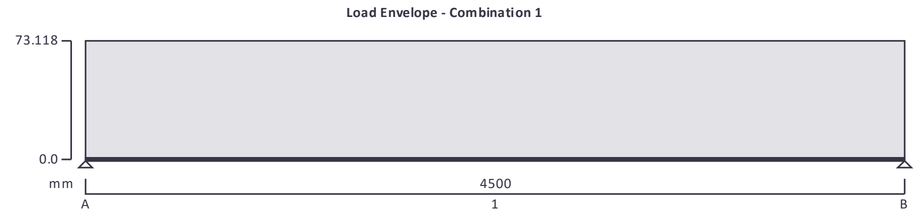
PASS - Maximum deflection does not exceed deflection limit

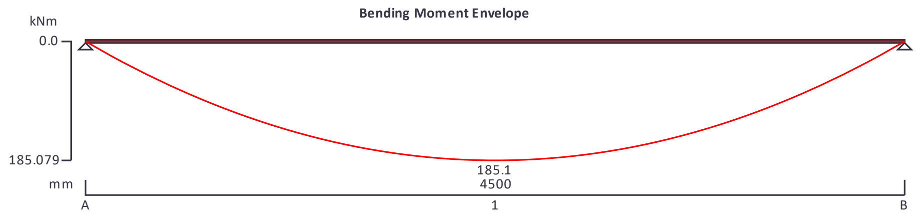
Tekla Tedds Form Structural Design 77 St John Street London EC1M 4NN	Project 24HEATH DRIVE				Job no. 162637	
	Calcs for BEAM G-14				Start page no./Revision 1	
	Calcs by CEM	Calcs date 15/02/2019	Checked by	Checked date	Approved by	Approved date

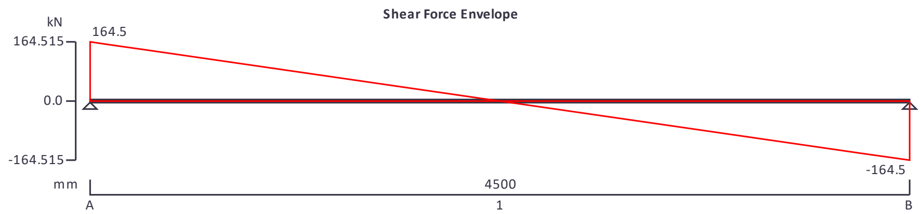
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 full UDL 29.4 kN/m Imposed full UDL 21.5 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35 Imposed × 1.50
		Permanent × 1.35 Imposed × 1.50
	Support B	Permanent × 1.35 Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **185.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **164.5** kN	V_{min} = **-164.5** kN
Deflection	δ_{max} = **9.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **164.5** kN	R_{A_min} = **164.5** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **68.1** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **48.4** kN	
Maximum reaction at support B	R_{B_max} = **164.5** kN	R_{B_min} = **164.5** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **68.1** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **48.4** kN	

Section details

Section type **UC 254x254x89 (BS4-1)**

Steel grade **S355**

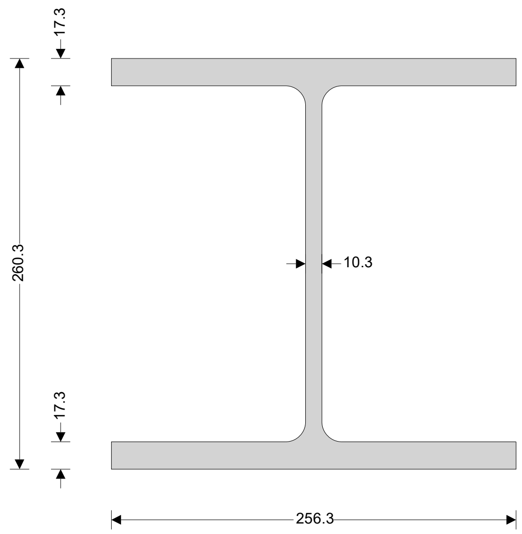
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = \max(t_f, t_w)$ = **17.3** mm

Nominal yield strength f_y = **345** N/mm^2

Nominal ultimate tensile strength f_u = **470** 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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.83**

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

Width of section — $c = d$ = **200.3** mm

$c / t_w = 23.6 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **110.3** mm

$c / t_f = 7.7 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **225.7** mm

Shear area factor — η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **164.5** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **3081** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **613.6** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **185.1** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **422.2** 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 — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.812**

Poissons ratio — ν = **0.3**

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

Unrestrained length — $L = 1.0 \times L_{s1}$ = **4500** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **1227.5** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.587**

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 — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.661**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.923**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.973**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.949**

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **400.7** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection δ_{lim} = min(10 mm, L_{s1} / 250) = **10** mm

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **9.226** mm

PASS - Maximum deflection does not exceed deflection limit

CALCULATION SHEET

JOB TITLE:	JOB NUMBER / FILE:	CALCULATION NUMBER:		Form
24 HEATH DRIVE	162637			
CALCULATION:	CALCULATION BY:	DATE:	CHECKED BY:	
G-14 LOAD TAKE DOWN	HD	07/12/18		

CALCULATIONS:

REF		OUTPUT

LOADING

ROOF LOAD

GK = 71.1 KN
QK = 40.2 KN

LOADS SPREAD OVER 4.6m OF WALL.

=> DL = 71.1kN/4.6m = 15.5kN/m
LL = 40.2kN/4.6m = 8.7 kN/m

TWO FLOORS OF TIMBER

DL = 1.02kN/m² × 8.5m/2 = 4.3 kN/m
LL = 1.5kN/m² × 8.5m/2 = 6.4 kN/m

×2
DL = 8.6kN/m
LL = 12.8kN/m

TWO FLOORS OF BRICK WORK WALL 7.5m

2.54 kN/m² × 7.5m = 19.1kN/m

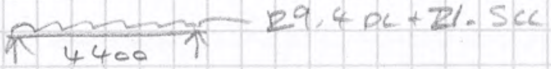
ONE FLOOR OF STUD WALL

0.55kN/m² × 3m = 1.7kN/m

TOTAL LOADS

DL = **29.4** kN/m
LL = 21.5 kN/m

CEM → G-14 BEAM



SEE TEDDS 254UC89

Mmax = 176.9 kNm
Vmax = 160.9kN
δmax $\frac{D+C}{250}$ = 8.4mm

$R_A = R_B$ = 160.9(66.6DL+47.3LL)kN

JOB TITLE:	JOB NUMBER / FILE:	CALCULATION NUMBER:		Form
24 HEATH DRIVE	162637			
CALCULATION:	CALCULATION BY:	DATE:	CHECKED BY:	
BEAM G-15, G-24, G-25 LOADING.	HD		ROM	

CALCULATIONS:

REF | OUTPUT

BEAM G-15

SUPPORTS GROUND FLOOR TIMBERS

DL : $1.02\ kN/m^2 \times 4.2m/2 = 2.1\ kN/m$

LL : $1.5\ kN/m^2 \times 4.2m/2 = 3.2\ kN/m$

BEAM G-24

SUPPORTS GROUND FLOOR AND
107mm THICK WALL.

WALL LOAD.
~~[illegible]~~ $2.54\ kN/m^2 \times 3m = 7.6\ kN/m$

FLOOR LOAD
DL : $1.02\ kN/m^2 \times 1m = 1.02\ kN/m$
LL : $1.5\ kN/m^2 \times 1m = 1.5\ kN/m$

TOTAL - DL : $8.6\ kN/m$
LL : $1.5\ kN/m$

BEAM G-25

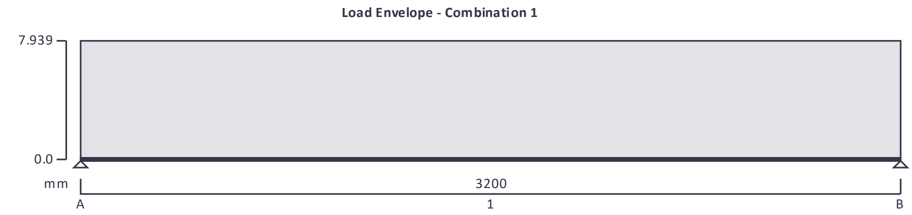
SUPPORTS GROUND FLOOR SLAB AT THE
FRONT OF THE PROPERTY.
200 THICK SLAB.
DL : $6.6\ kN/m^2 \times 2m = 13.2\ kN/m$
LL : $1.5\ kN/m^2 \times 2m = 3\ kN/m$

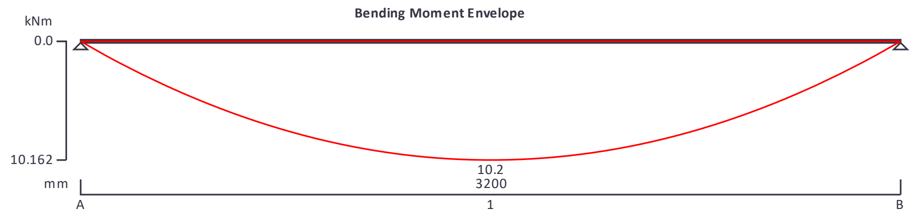
Tekla Tedds Form Structural Design 77 St John Street London EC1M 4NN	Project 24 HEATH DRIVE				Job no. 162637	
	Calcs for BEAM G-15				Start page no./Revision 1	
	Calcs by HD	Calcs date 13/02/2019	Checked by ROM	Checked date	Approved by	Approved date

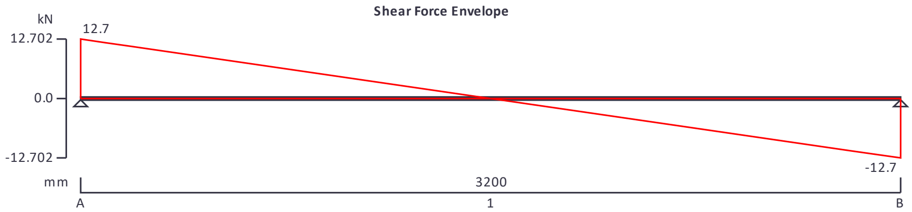
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 full UDL 2.1 kN/m
	Imposed full UDL 3.2 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Imposed × 1.50
		Permanent × 1.35
		Imposed × 1.50
	Support B	Permanent × 1.35
		Imposed × 1.50

Tekla Tedds Form Structural Design 77 St John Street London EC1M 4NN	Project 24 HEATH DRIVE				Job no. 162637	
	Calcs for BEAM G-15				Start page no./Revision 2	
	Calcs by HD	Calcs date 13/02/2019	Checked by ROM	Checked date	Approved by	Approved date

Analysis results

Maximum moment	M_{max} = **10.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **12.7** kN	V_{min} = **-12.7** kN
Deflection	δ_{max} = **2.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **12.7** kN	R_{A_min} = **12.7** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **3.7** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **5.1** kN	
Maximum reaction at support B	R_{B_max} = **12.7** kN	R_{B_min} = **12.7** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **3.7** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **5.1** kN	

Section details

Section type — **UC 152x152x23 (BS4-1)**

Steel grade — **S355**

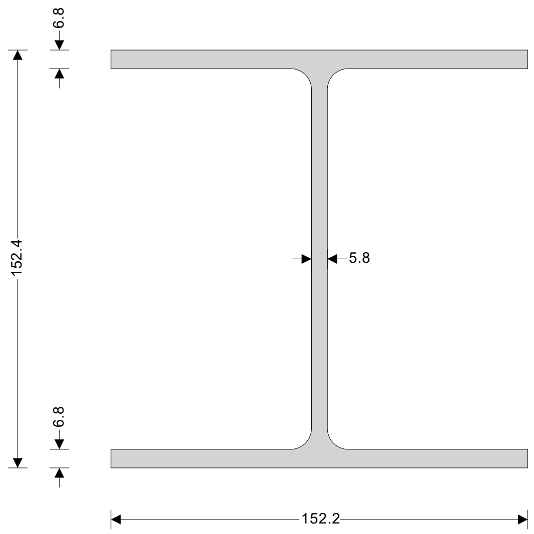
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element — $t = \max(t_f, t_w)$ = **6.8** mm

Nominal yield strength — f_y = **355** N/mm^2

Nominal ultimate tensile strength — f_u = **470** 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

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = \mathbf{0.81}$

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

Width of section — $c = d = \mathbf{123.6}$ mm

$c / t_w = 26.2 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 11.9 \times \varepsilon <= 14 \times \varepsilon$ — Class 3

Section is class 3

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{138.8}$ mm

Shear area factor — $\eta = \mathbf{1.000}$

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{12.7}$ kN

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

Design shear resistance - cl 6.2.6(2) — $V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{204.4}$ kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{10.2}$ kNm

Design bending resistance moment - eq 6.14 — $M_{c,Rd} = M_{el,Rd} = W_{el.y} \times f_y / \gamma_{M0} = \mathbf{58.2}$ kNm

Slenderness ratio for lateral torsional buckling

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

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

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.825}$

Poissons ratio — $\nu = \mathbf{0.3}$

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

Unrestrained length — $L = 1.0 \times L_{s1} = \mathbf{3200}$ mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{110.7}$ kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{el.y} \times f_y / M_{cr})} = \mathbf{0.725}$

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

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

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

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{0.753}$

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.857}$

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

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.883}$

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{el.y} \times f_y / \gamma_{M1} = \mathbf{51.4}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection δ_{lim} = min(10 mm, L_{s1} / 250) = **10** mm

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **2.874** mm

PASS - Maximum deflection does not exceed deflection limit

JOB TITLE:	JOB NUMBER / FILE:	CALCULATION NUMBER:		Form
24 Heath Drive	162637			
CALCULATION:	CALCULATION BY:	DATE:	CHECKED BY:	
BEAM G-16 LOADING	HD	16/01/19	ROM	

CALCULATIONS:

REF | OUTPUT

GROUND FLOOR

TIMBER FLOOR SPANS 1.5m
DL = 1.02 kN/m² × 1.5m/2 = 0.8 kN/m
LL = 1.5 kN/m² × 1.5m/2 = 1.1 kN/m

STAIRS LOAD
TIMBER STAIRS SPANNING 3m
DL = 1.02 kN/m² × 3m/2 = 1.6 kN/m
LL = 1.5 kN/m² × 3m/2 = 2.2 kN/m

1st FLOOR
STAIRS LOAD SPANNING 3M
DL = 1.02 kN/m² × 3m/2 = 1.6 kN/m
LL = 1.5 kN/m² × 3m/2 = 2.2 kN/m

TIMBER FLOOR, 1m SPAN CONSIDERED.
DL = 1.02 kN/m² × 1m = 1.02 kN/m
LL = 1.5 kN/m² × 1m = 2.2 kN/m

2ND FLOOR
TIMBER FLOOR SPANS 5m +1kN/m FOR PARTITIONS
DL = 2.02 kN/m² × 5m/2 = 5.1 kN/m
LL = 1.5 kN/m² × 5m/2 = 3.8 kN/m

WALL LOAD, WALL IS 100mm thick
DL = 2.54 kN/m² × 6.6m = 16.8 kN/m

ROOF LOAD
FROM ROOF DESIGN CALCULATIONS.

POINT LOADS. DL = 58.2 kN
LL = 28.9 kN

and DL = 37.3 kN
LL = 28.2 kN

THESE LOADS ARE SPREAD ACROSS THE WALL. (4.5m)

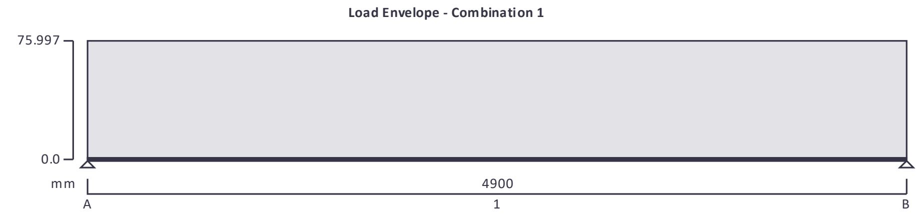
$\Rightarrow$ DL = (58.2+37.3)/6 ≅ 16 kN/m
LL = (28.9+28.2)/6 ≅ 10 kN/m

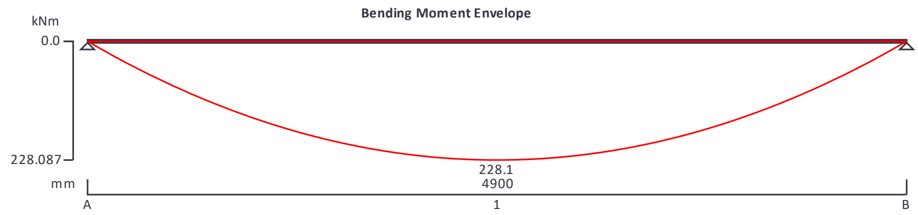
TOTAL LOADS
DEAD LOAD = 43 kN/m
LIVE LOAD = 38 kN/m

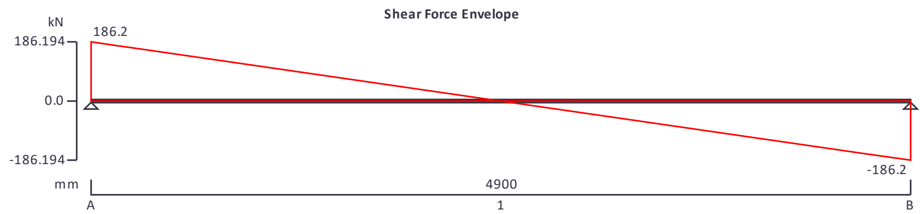
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 full UDL 43 kN/m
	Imposed full UDL 10.8 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Imposed × 1.50
		Permanent × 1.35
		Imposed × 1.50
	Support B	Permanent × 1.35
		Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **228.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **186.2** kN	V_{min} = **-186.2** kN
Deflection	δ_{max} = **8.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **186.2** kN	R_{A_min} = **186.2** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **108.5** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **26.5** kN	
Maximum reaction at support B	R_{B_max} = **186.2** kN	R_{B_min} = **186.2** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **108.5** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **26.5** kN	

Section details

Section type **UC 254x254x132**

Steel grade **S355**

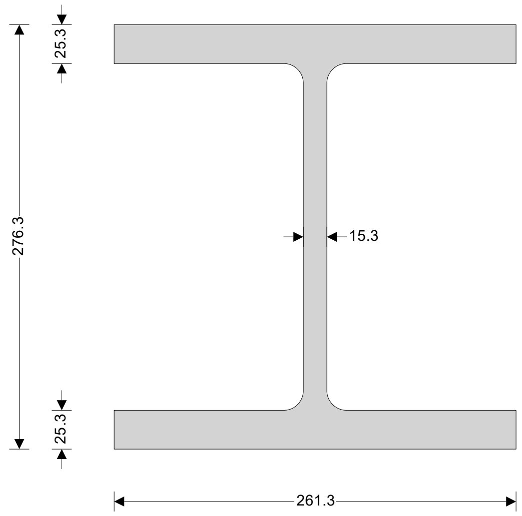
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = \max(t_f, t_w)$ = **25.3** mm

Nominal yield strength f_y = **345** N/mm^2

Nominal ultimate tensile strength f_u = **470** 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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \mathbf{0.83}$

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

Width of section — $c = d = \mathbf{200.3}$ mm

$c / t_w = 15.9 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2 = \mathbf{110.3}$ mm

$c / t_f = 5.3 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{225.7}$ mm

Shear area factor — $\eta = \mathbf{1.000}$

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{186.2}$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{4621}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{920.5}$ kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{228.1}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{644.9}$ kNm

Slenderness ratio for lateral torsional buckling

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

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

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.816}$

Poissons ratio — $\nu = \mathbf{0.3}$

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

Unrestrained length — $L = 1.0 \times L_{s1} = \mathbf{4900}$ mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} =$ **2121.2** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{0.551}$

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

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

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

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{0.640}$

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.939}$

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.974}$

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.964}$

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{621.6}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection δ_{lim} = min(10 mm, L_{s1} / 250) = **10** mm

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **8.741** mm

PASS - Maximum deflection does not exceed deflection limit

JOB TITLE:	JOB NUMBER / FILE:	CALCULATION NUMBER:		Form
24 HEATH DRIVE				
CALCULATION:	CALCULATION BY:	DATE:	CHECKED BY:	
G-17, G-22, G-23, G-18	HD			

CALCULATIONS:

REF | OUTPUT

G-17 LOAD

SPAN = 4.6 m

LOADING

TIMBER FLOOR

DL: (1.02 kN/m² X 3m) = 3.06 kN/m
LL: (1.5 kN/m² x 3m) = 4.5 kN/m

2M OF 0.1M WIDE BRICKWALL

DL: 0.1 m x 2m x 20kN/m³ = 4kN/m

TOTAL: DL = 7.06 kN/m
LL = 4.5 kN/m

350 deep x 300 wide RC pynford

G-22 & G-23, SPAN 4.6M

LOADING

MASONARY WALL.
DL: 3.3m x 0.1m x 20kN/m³ = 6.6 kN/m

TIMBER FLOOR. 1.5M SPAN x 2 (2 floors)
DL: 1.02 kN/m² x 1.5m = 1.53 kN/m
LL: 1.5 kN/m² x 1.5m = 2.25 kN/m

TOTAL: DL = 6.6 kN/m + (1.53x2) = 9.7 kN/m
LL = 2.25 kN/m x 2 = 4.5 kN/m

350 deep x 300 wide RC PYNFORD.

G-18 LOAD

G22 & G23 REACTIONS

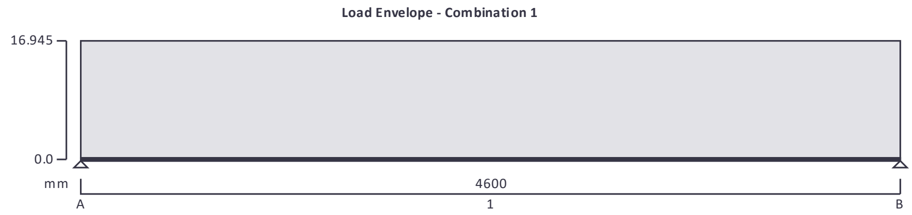
Gk = 30.4 kN
Qk = 10.8 kN

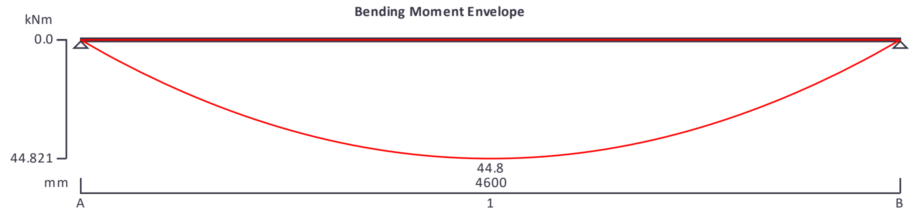
BRICK WORK PIERS AT EITHER END & HEIGHT = 3.3M
100mm thick.
DL: 0.1m x 3.3m x 20kN/m³ = 6.6 kN/m

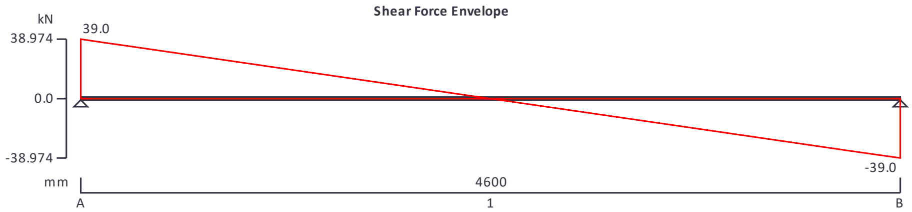
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 full UDL 7.1 kN/m
	Imposed full UDL 4.5 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Imposed × 1.50
		Permanent × 1.35
		Imposed × 1.50
	Support B	Permanent × 1.35
		Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **44.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **39** kN	V_{min} = **-39** kN
Deflection	δ_{max} = **7.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **39** kN	R_{A_min} = **39** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **17.4** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **10.4** kN	
Maximum reaction at support B	R_{B_max} = **39** kN	R_{B_min} = **39** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **17.4** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **10.4** kN	

Section details

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

Steel grade — **S355**

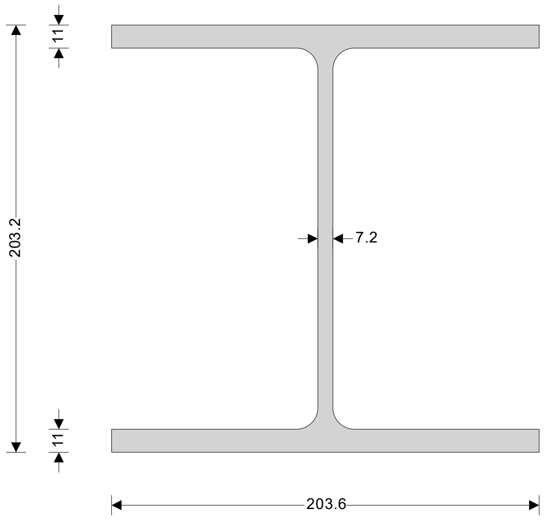
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element — $t = \max(t_f, t_w)$ = **11.0** mm

Nominal yield strength — f_y = **355** N/mm^2

Nominal ultimate tensile strength — f_u = **470** 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

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]}$ = **0.81**

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

Width of section $c = d$ = **160.8** mm

$c / t_w = 27.4 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **181.2** mm

Shear area factor η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **39** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **347.9** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **44.8** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **176.6** 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 $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.813**

Poissons ratio ν = **0.3**

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

Unrestrained length $L = 1.0 \times L_{s1}$ = **4600** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **306.1** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.759**

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 $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.777**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.839**

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 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.865**

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **152.7** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection δ_{lim} = min(10 mm, L_{s1} / 250) = **10** mm

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **7.325** mm

PASS - Maximum deflection does not exceed deflection limit

Tekla Tedds Form Structural Design 77 St John Street London EC1M 4NN	Project 24 HEATH DRIVE				Job no. 162637	
	Calcs for BEAM G-18				Start page no./Revision 1	
	Calcs by HD	Calcs date 15/01/2019	Checked by ROM	Checked date	Approved by	Approved date

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

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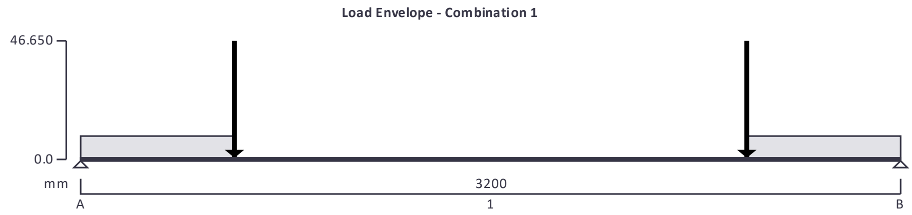
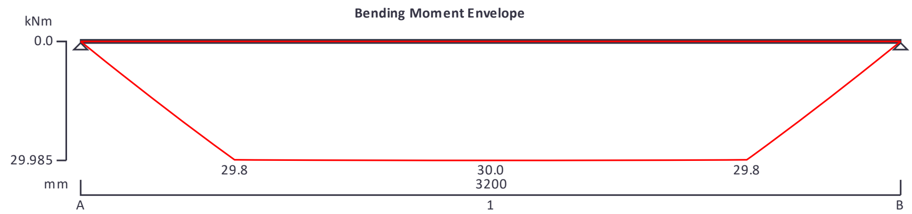
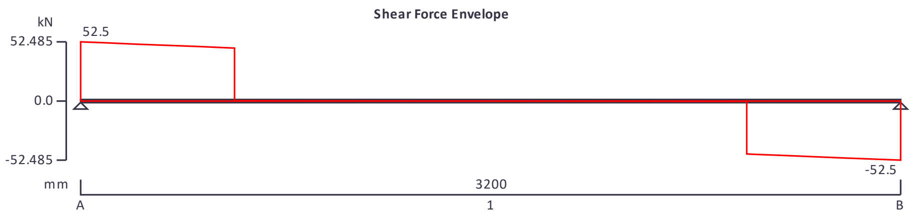
TEDDS calculation version 3.0.13







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 23 kN at 600 mm
	Imposed point load 10.4 kN at 600 mm
	Permanent point load 23 kN at 2600 mm
	Imposed point load 10.4 kN at 2600 mm
	Permanent partial UDL 6.6 kN/m from 0 mm to 600 mm
	Permanent partial UDL 6.6 kN/m from 2600 mm to 3200 mm

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Imposed × 1.50

		Permanent × 1.35
		Imposed × 1.50
	Support B	Permanent × 1.35
		Imposed × 1.50

Analysis results

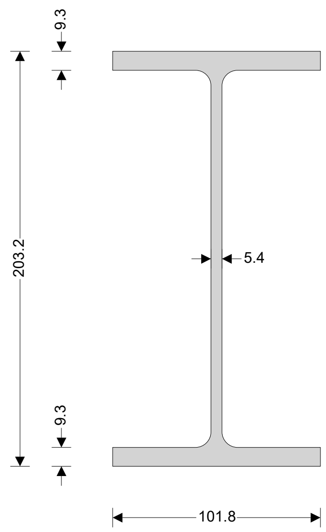
Maximum moment	M_{max} = **30** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **52.5** kN	V_{min} = **-52.5** kN
Deflection	δ_{max} = **5.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **52.5** kN	R_{A_min} = **52.5** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **27.3** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **10.4** kN	
Maximum reaction at support B	R_{B_max} = **52.5** kN	R_{B_min} = **52.5** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **27.3** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **10.4** kN	

Section details

Section type	**UB 203x102x23 (BS4-1)**
Steel grade	**S355**

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 = **355** N/mm^2
Nominal ultimate tensile strength	f_u = **470** 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

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]}$ = **0.81**

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

Width of section $c = d$ = **169.4** mm

$c / t_w = 38.6 \times \varepsilon <= 72 \times \varepsilon$ 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

$c / t_f = 5.4 \times \varepsilon <= 9 \times \varepsilon$ 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 $V_{Ed} = \max(abs(V_{max}), abs(V_{min}))$ = **52.5** 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) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **253.7** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment $M_{Ed} = \max(abs(M_{s1\_max}), abs(M_{s1\_min}))$ = **30** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **83.1** 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 $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.96**

Poissons ratio ν = **0.3**

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

Unrestrained length $L = 1.0 \times L_{s1}$ = **3200** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **63.6** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.143**

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 $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.116**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.613**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.977**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.627**

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **52.1** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection δ_{lim} = min(10 mm, L_{s1} / 250) = **10** mm

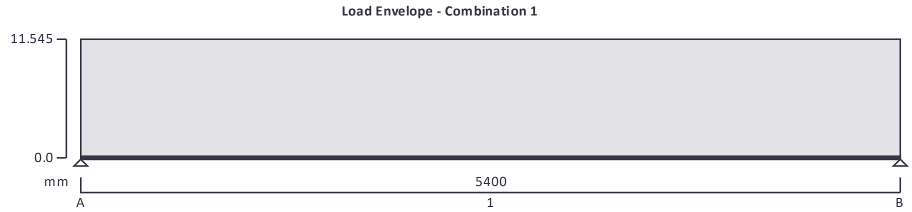
Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **5.937** mm

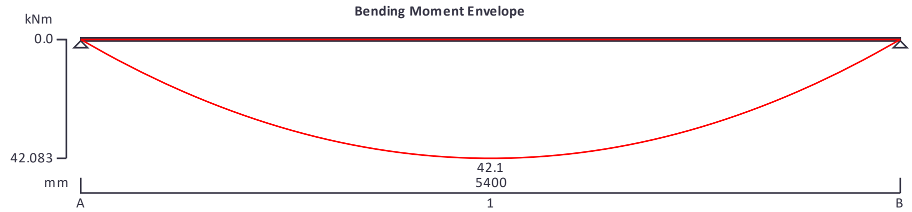
PASS - Maximum deflection does not exceed deflection limit

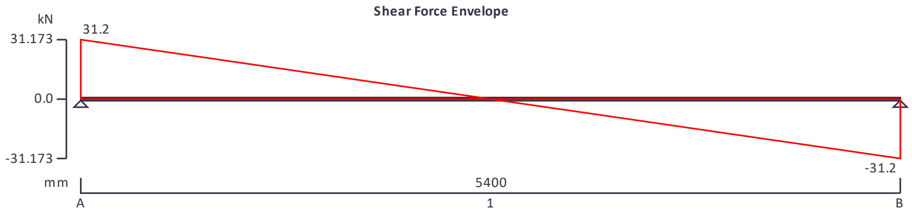
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 full UDL 3.1 kN/m
	Imposed full UDL 4.5 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Imposed × 1.50
		Permanent × 1.35
		Imposed × 1.50
	Support B	Permanent × 1.35
		Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **42.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **31.2** kN	V_{min} = **-31.2** kN
Deflection	δ_{max} = **9.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **31.2** kN	R_{A_min} = **31.2** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **9.6** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **12.2** kN	
Maximum reaction at support B	R_{B_max} = **31.2** kN	R_{B_min} = **31.2** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **9.6** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **12.2** kN	

Section details

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

Steel grade **S355**

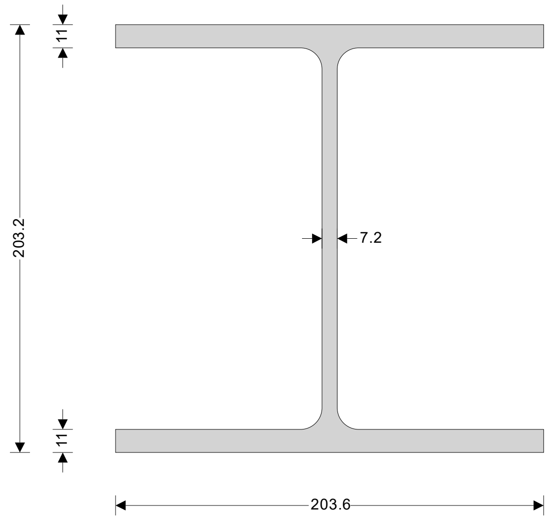
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = \max(t_f, t_w)$ = **11.0** mm

Nominal yield strength f_y = **355** N/mm^2

Nominal ultimate tensile strength f_u = **470** 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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

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

Width of section $c = d =$ **160.8** mm

$c / t_w = 27.4 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 =$ **88** mm

$c / t_f = 9.8 \times \varepsilon <= 10 \times \varepsilon$ Class 2

Section is class 2

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f =$ **181.2** mm

Shear area factor $\eta =$ **1.000**

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

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **31.2** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) =$ **1698** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} =$ **347.9** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **42.1** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} =$ **176.6** 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 $g = \sqrt{[1 - (I_z / I_y)]} =$ **0.813**

Poissons ratio $\nu =$ **0.3**

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

Unrestrained length $L = 1.0 \times L_{s1} =$ **5400** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} =$ **244.6** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} =$ **0.85**

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 $\alpha_{LT} =$ **0.34**

Correction factor for rolled sections $\beta =$ **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] =$ **0.847**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) =$ **0.789**

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 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) =$ **0.814**

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} =$ **143.7** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection $\delta_{lim} = \min(10 \text{ mm}, L_{s1} / 250) =$ **10** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **9.294** mm

PASS - Maximum deflection does not exceed deflection limit

CALCULATION SHEET

JOB TITLE:	JOB NUMBER / FILE:	CALCULATION NUMBER:		Form
24 HEATH DRIVE	162637			
CALCULATION:	CALCULATION BY:	DATE:	CHECKED BY:	
G-20 LOADING	HD	04/12/18		

CALCULATIONS:

REF | OUTPUT

G-20 LOADING.

ROOF LOAD 1.6m of Roof, 1.53kN/m²

DL = 1.6 x 1.53kN/m² = 2.4 kN/m
LL = 1.6m x 0.6 kN/m² = 1 kN/m

SECOND FLOOR LOAD
TIMBER Floor SPAN 2M, 1.02 kN/m²

DL = 2m x 1.02kN/m² = 2.04 kN/m
LL = 2m x 1.5 kN/m² = 3 kN/m

FIRST FLOOR LOAD
TIMBER FLOOR SPAN 4.1M, 1.02 kN/m²

DL = 4.1 m x 1.02 kN/m² = 4.2 kN/m
LL = 4.1m x 1.5 kN/m² = 6.2 kN/m

WALL LOADS

3.2m of BRICK WALL, 100mm thick, 2.54 kN/m²

DL = 3.2m x 2.54 kN/m² = 8.1 kN/m
2.9m of STUD WALL. DL = 2.9m x 0.55kN/m² = 1.6

TOTAL LOADS ACTING UPON EXISTING
FIRST FLOOR BEAM 1

DL = 16.74 kN/m
LL = 13.6 kN/m

BEAM SPAN = 3.5m
SUPPORT REACTIONS: Gk = 30 KN
Qk = 23.8 KN

TOTAL LOADS ACTING UPON EXISTING FIRST floor
BEAM 2
4m of FLOOR 2 LEVELS AND ONE LEVEL OF
STUD WALL.

DL = [4m x 1.02kN/m² x 2] + [0.55 x 3.2m] = 10kN/m
LL = [4m x 1.5kN/m² x 2] = 12 kN/m
BEAM SPAN = 3.1m

SUPPORT REACTIONS: Gk = 16 KN
Qk = 18.6 kN

CALCULATION SHEET

JOB TITLE:	JOB NUMBER / FILE:	CALCULATION NUMBER:		Form
24 HEATH DRIVE				
CALCULATION:	CALCULATION BY:	DATE:	CHECKED BY:	
G-20 LOADING CONTINUED.	HD	04/12/18		

CALCULATIONS:

REF | OUTPUT

G-20 LOADING CONTINUED.

GROUND FLOOR LOAD 2.6m SPAN.

2.6m X 1.02 KN/m² = 2.7 KN/m
2.6m X 1.5 KN/m² = 3.9 KN/m

BEAM 1 LOAD @ 1.2m & 4.5m
GK = 30.0 KN
QK = 23.8 KN

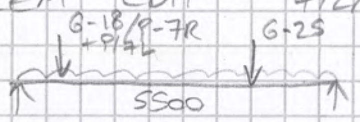
BEAM 2 LOAD @ 4.5M
GK = 16.1 KN
QK = 18.6 KN

G-17 & G-18 ALSO ACT UPON BEAM G-20

P7LAS STEEL REACTIONS GK = 17.4 KN
QK = 10.4 KN

P7R AS STEEL REACTIONS GK = 27.3 KN
QK = 10.4 KN

CEM EDIT 4/2/19



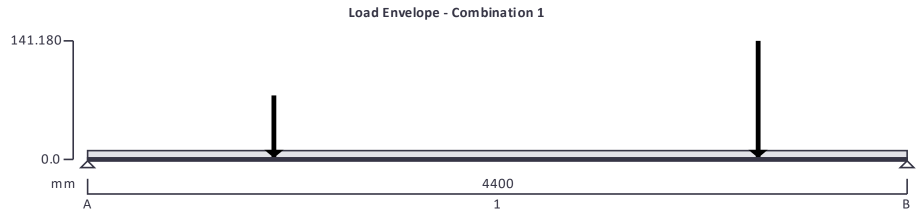
ASSUMED G-25 SIMILAR TO G-18

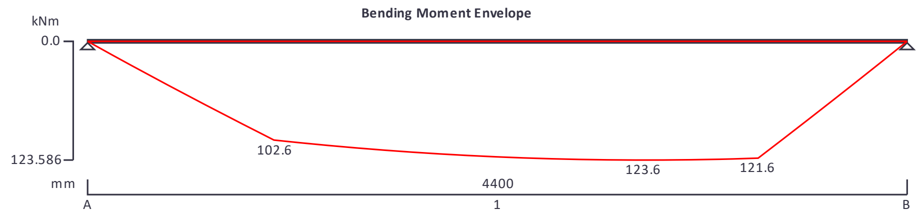
G-18 + G-17 @ 1224 = (17.4DL + 10.4LL) + (27.3DL + 10.4LL)
G-25 @ 4350 = 27.3DL + 10.4LL

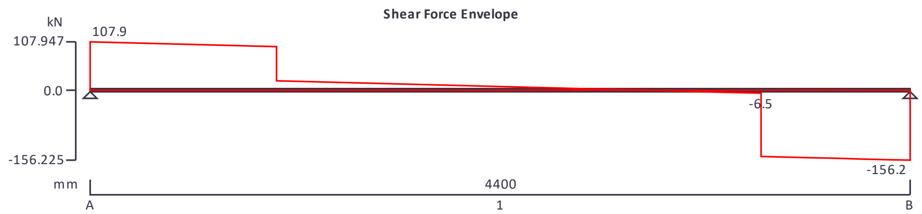
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 full UDL 2.7 kN/m
	Permanent point load 30 kN at 1000 mm
	Permanent point load 60.8 kN at 3600 mm
	Imposed full UDL 3.9 kN/m
	Imposed point load 23.8 kN at 1000 mm
	Imposed point load 39.4 kN at 3600 mm

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Imposed × 1.50

Permanent × 1.35

Imposed × 1.50

Support B — Permanent × 1.35

Imposed × 1.50

Analysis results

Maximum moment — M_{max} = **123.6** kNm — M_{min} = **0** kNm

Maximum shear — V_{max} = **107.9** kN — V_{min} = **-156.2** kN

Deflection — δ_{max} = **9.6** mm — δ_{min} = **0** mm

Maximum reaction at support A — R_{A_max} = **107.9** kN — R_{A_min} = **107.9** kN

Unfactored permanent load reaction at support A — $R_{A\_Permanent}$ = **42** kN

Unfactored imposed load reaction at support A — $R_{A\_Imposed}$ = **34.1** kN

Maximum reaction at support B — R_{B_max} = **156.2** kN — R_{B_min} = **156.2** kN

Unfactored permanent load reaction at support B — $R_{B\_Permanent}$ = **64.4** kN

Unfactored imposed load reaction at support B — $R_{B\_Imposed}$ = **46.2** kN

Section details

Section type — **UC 203x203x86 (BS4-1)**

Steel grade — **S355**

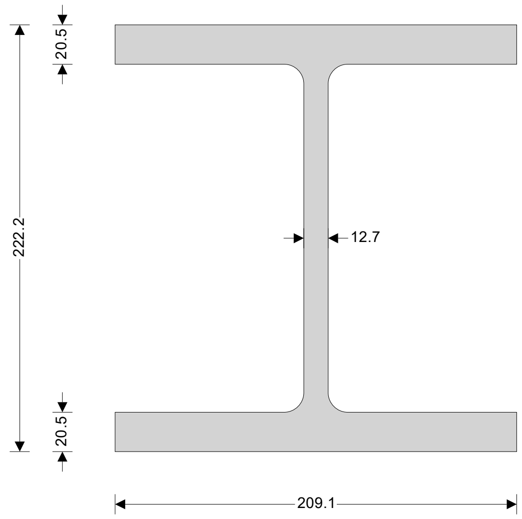
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element — $t = max(t_f, t_w)$ = **20.5** mm

Nominal yield strength — f_y = **345** N/mm^2

Nominal ultimate tensile strength — f_u = **470** 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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.83**

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

Width of section $c = d$ = **160.8** mm

$c / t_w = 15.3 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 5.2 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **181.2** mm

Shear area factor η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **156.2** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **3069** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **611.3** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **123.6** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **337** 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 $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.818**

Poissons ratio ν = **0.3**

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

Unrestrained length $L = 1.0 \times L_{s1}$ = **4400** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **962.6** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.592**

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 $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.664**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.921**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.973**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.947**

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **319** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection δ_{lim} = min(10.04 mm, L_{s1} / 250) = **10** mm

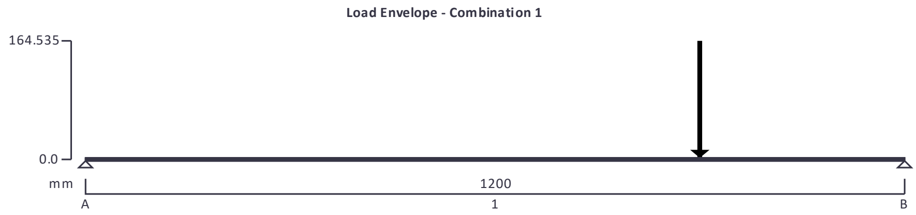
Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **9.566** mm

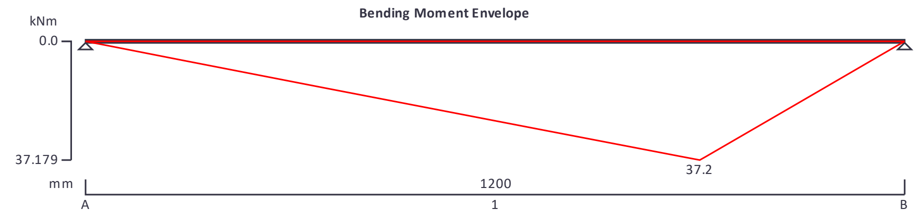
PASS - Maximum deflection does not exceed deflection limit

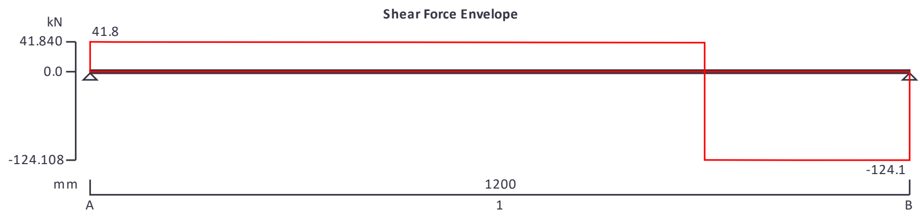
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 68.1 kN at 900 mm
	Imposed point load 48.4 kN at 900 mm

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Imposed × 1.50
		Permanent × 1.35
		Imposed × 1.50
	Support B	Permanent × 1.35
		Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **37.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **41.8** kN	V_{min} = **-124.1** kN
Deflection	δ_{max} = **0.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **41.8** kN	R_{A_min} = **41.8** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **17.5** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **12.1** kN	
Maximum reaction at support B	R_{B_max} = **124.1** kN	R_{B_min} = **124.1** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **51.6** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **36.3** kN	

Section details

Section type — **UC 254x254x89 (BS4-1)**

Steel grade — **S355**

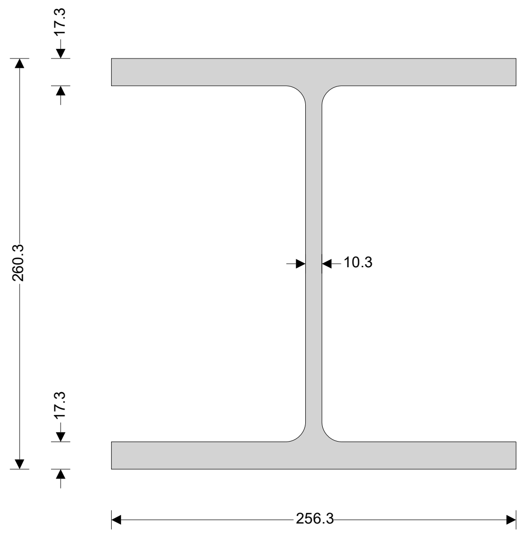
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element — $t = \max(t_f, t_w)$ = **17.3** mm

Nominal yield strength — f_y = **345** N/mm^2

Nominal ultimate tensile strength — f_u = **470** 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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.83**

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

Width of section — $c = d$ = **200.3** mm

$c / t_w = 23.6 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **110.3** mm

$c / t_f = 7.7 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **225.7** mm

Shear area factor — η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **124.1** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **3081** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **613.6** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **37.2** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **422.2** 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 — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.812**

Poissons ratio — ν = **0.3**

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

Unrestrained length — $L = 1.0 \times L_{s1}$ = **1200** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **12302.5** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.185**

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

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

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection — $\delta_{lim} = L_{s1} / 250$ = **4.8** mm

Maximum deflection span 1 — $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **0.099** mm

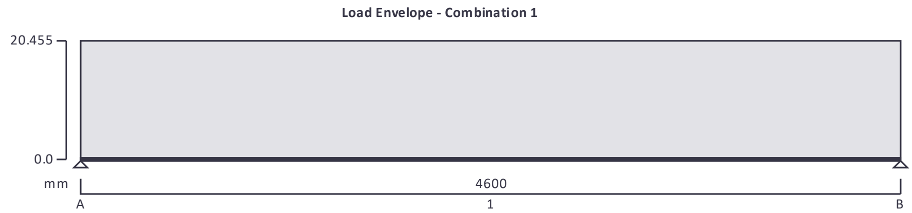
PASS - Maximum deflection does not exceed deflection limit

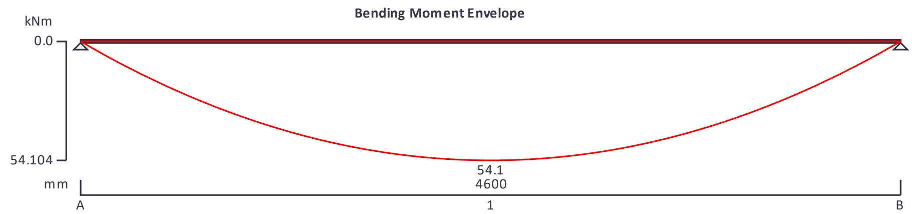
Tekla Tedds	Project				Job no.	
Form Structural Design 77 St John Street London EC1M 4NN	24 HEATH DRIVE					
	Calcs for				Start page no./Revision	
	BEAM G-22 AND G-23				1	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	HD	16/01/2019	ROM			

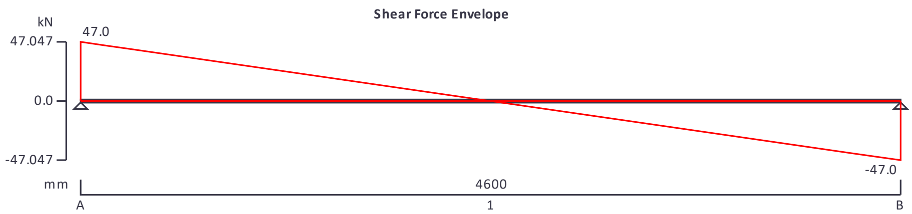
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 full UDL 9.7 kN/m
	Imposed full UDL 4.5 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Imposed × 1.50
		Permanent × 1.35
		Imposed × 1.50
	Support B	Permanent × 1.35
		Imposed × 1.50

Analysis results

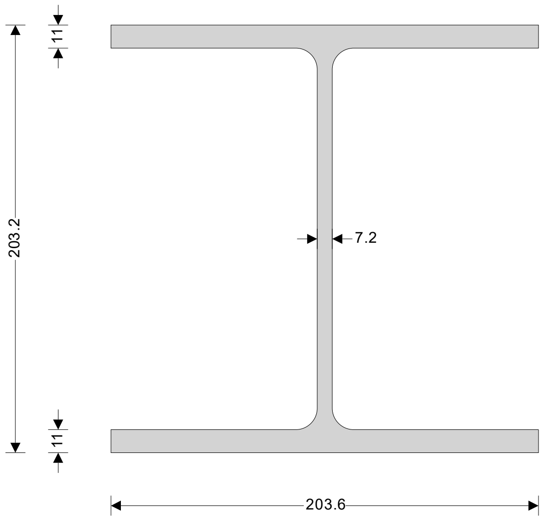
Maximum moment	M_{max} = **54.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **47** kN	V_{min} = **-47** kN
Deflection	δ_{max} = **8.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **47** kN	R_{A_min} = **47** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **23.3** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **10.4** kN	
Maximum reaction at support B	R_{B_max} = **47** kN	R_{B_min} = **47** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **23.3** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **10.4** kN	

Section details

Section type	**UC 203x203x46 (BS4-1)**
Steel grade	**S355**

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

Nominal thickness of element	$t = max(t_f, t_w)$ = **11.0** mm
Nominal yield strength	f_y = **355** N/mm^2
Nominal ultimate tensile strength	f_u = **470** 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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

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

Width of section — $c = d$ = **160.8** mm

$c / t_w = 27.4 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 9.8 \times \varepsilon <= 10 \times \varepsilon$ — Class 2

Section is class 2

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **181.2** mm

Shear area factor — η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **47** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **347.9** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **54.1** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **176.6** 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 — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.813**

Poissons ratio — ν = **0.3**

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

Unrestrained length — $L = 1.0 \times L_{s1}$ = **4600** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **306.1** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.759**

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 — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.777**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.839**

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 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.865**

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **152.7** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection δ_{lim} = min(10 mm, L_{s1} / 250) = **10** mm

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **8.905** mm

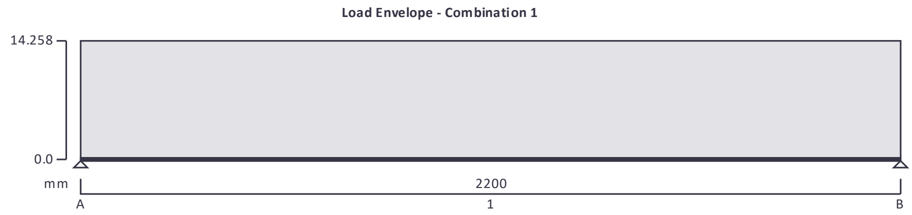
PASS - Maximum deflection does not exceed deflection limit

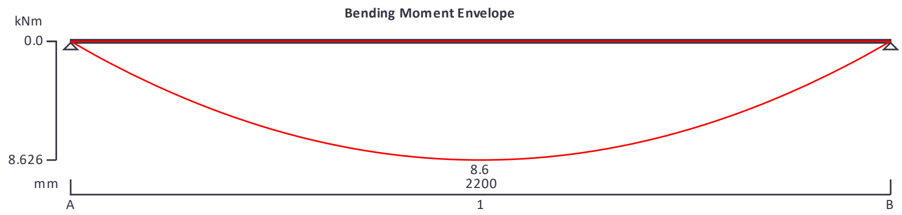
Tekla Tedds Form Structural Design 77 St John Street London EC1M 4NN	Project 24 HEATH DRIVE				Job no. 162637	
	Calcs for BEAM G-24				Start page no./Revision 1	
	Calcs by HD	Calcs date 13/02/2019	Checked by ROM	Checked date	Approved by	Approved date

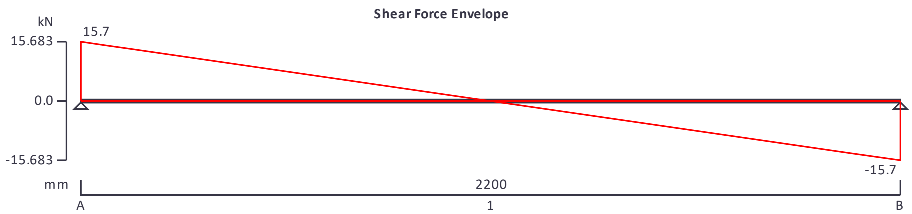
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 full UDL 8.6 kN/m
	Imposed full UDL 1.5 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Imposed × 1.50
		Permanent × 1.35
		Imposed × 1.50
	Support B	Permanent × 1.35
		Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **8.6** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **15.7** kN	V_{min} = **-15.7** kN
Deflection	δ_{max} = **0.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **15.7** kN	R_{A_min} = **15.7** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **9.8** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **1.7** kN	
Maximum reaction at support B	R_{B_max} = **15.7** kN	R_{B_min} = **15.7** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **9.8** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **1.7** kN	

Section details

Section type — **UC 152x152x30 (BS4-1)**

Steel grade — **S355**

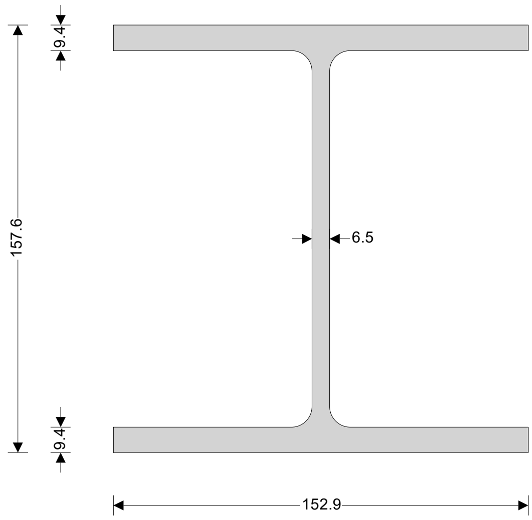
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element — $t = \max(t_f, t_w)$ = **9.4** mm

Nominal yield strength — f_y = **355** N/mm^2

Nominal ultimate tensile strength — f_u = **470** 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

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} = \mathbf{0.81}$

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

Width of section — $c = d = \mathbf{123.6}$ mm

$c / t_w = 23.4 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

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

$c / t_f = 8.6 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f = \mathbf{138.8}$ mm

Shear area factor — $\eta = \mathbf{1.000}$

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{15.7}$ kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{1156}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{236.9}$ kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{8.6}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0} = \mathbf{87.9}$ kNm

Slenderness ratio for lateral torsional buckling

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

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

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.824}$

Poissons ratio — $\nu = \mathbf{0.3}$

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

Unrestrained length — $L = 1.0 \times L_{s1} = \mathbf{2200}$ mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{313.2}$ kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})} = \mathbf{0.53}$

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

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

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

LTB reduction determination factor — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{0.627}$

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.948}$

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.974}$

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.973}$

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1} = \mathbf{85.5}$ kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection δ_{lim} = min(10 mm, L_{s1} / 250) = **8.8** mm

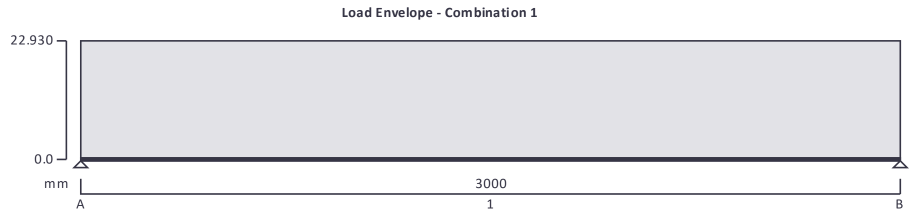
Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **0.864** mm

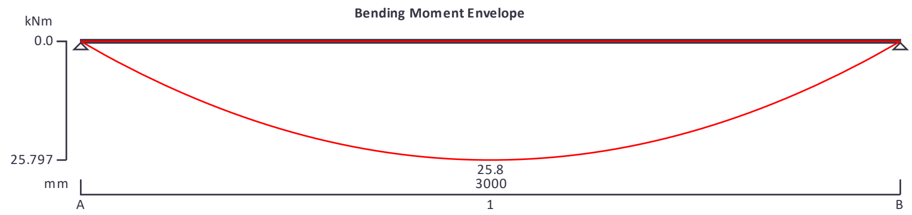
PASS - Maximum deflection does not exceed deflection limit

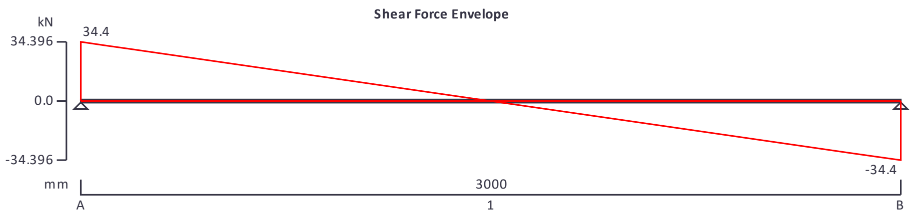
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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 full UDL 13.2 kN/m Imposed full UDL 3 kN/m

Load combinations

Load combination 1	Support A	Permanent × 1.35 Imposed × 1.50
		Permanent × 1.35 Imposed × 1.50
	Support B	Permanent × 1.35 Imposed × 1.50

Analysis results

Maximum moment	M_{max} = **25.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **34.4** kN	V_{min} = **-34.4** kN
Deflection	δ_{max} = **1.8** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **34.4** kN	R_{A_min} = **34.4** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **20.5** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **4.5** kN	
Maximum reaction at support B	R_{B_max} = **34.4** kN	R_{B_min} = **34.4** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **20.5** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **4.5** kN	

Section details

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

Steel grade **S355**

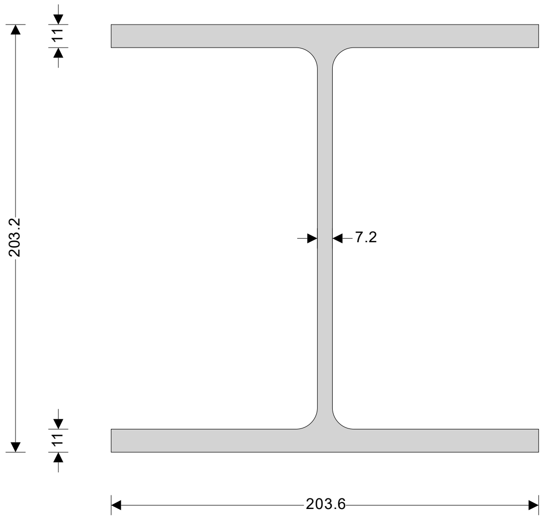
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = \max(t_f, t_w)$ = **11.0** mm

Nominal yield strength f_y = **355** N/mm^2

Nominal ultimate tensile strength f_u = **470** 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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.81**

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

Width of section — $c = d$ = **160.8** mm

$c / t_w = 27.4 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **88** mm

$c / t_f = 9.8 \times \varepsilon <= 10 \times \varepsilon$ — Class 2

Section is class 2

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **181.2** mm

Shear area factor — η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **34.4** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1698** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **347.9** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **25.8** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **176.6** 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 — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.813**

Poissons ratio — ν = **0.3**

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

Unrestrained length — $L = 1.0 \times L_{s1}$ = **3000** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **592.5** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.546**

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 — $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.637**

LTB reduction factor - eq 6.57 — $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.941**

Modification factor — $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.974**

Modified LTB reduction factor - eq 6.58 — $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.966**

Design buckling resistance moment - eq 6.55 — $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **170.6** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and imposed loads

Limiting deflection δ_{lim} = min(10 mm, L_{s1} / 250) = **10** mm

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **1.831** mm

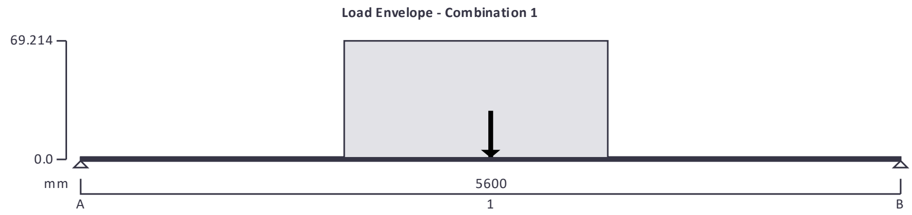
PASS - Maximum deflection does not exceed deflection limit

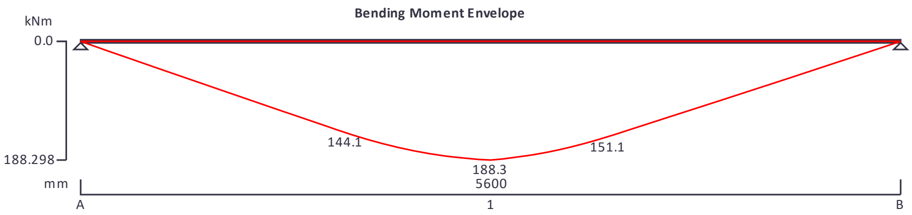
Tekla Tedds Form Structural Design 77 St John Street London EC1M 4NN	Project 24 HEATH DRIVE				Job no. 162637	
	Calcs for BEAM G-27				Start page no./Revision 1	
	Calcs by ROM	Calcs date 06/02/2019	Checked by	Checked date	Approved by	Approved date

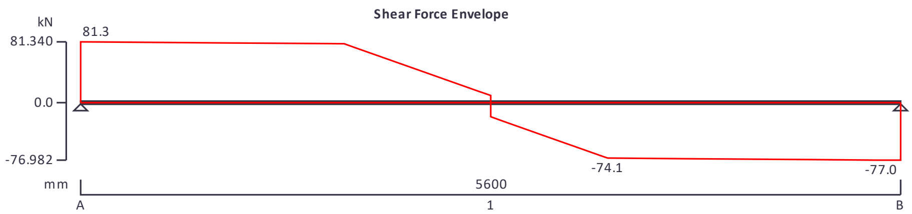
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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
	G-7 - Permanent point load 21 kN at 2800 mm
	CHIMNEY BREAST - Permanent partial UDL 50.22 kN/m from 1800 mm to 3600 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} = **188.3** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **188.3** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **188.3** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum shear	V_{max} = **81.3** kN	V_{min} = **-77** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **81.3** kN	$V_{s1\_seg1\_min}$ = **-18.8** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **0** kN	$V_{s1\_seg2\_min}$ = **-77** kN
Deflection segment 3	δ_{max} = **11** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **81.3** kN	R_{A_min} = **81.3** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **60.3** kN	
Maximum reaction at support B	R_{B_max} = **77** kN	R_{B_min} = **77** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **57** kN	

Section details

Section type **UC 254x254x107 (BS4-1)**

Steel grade **S355**

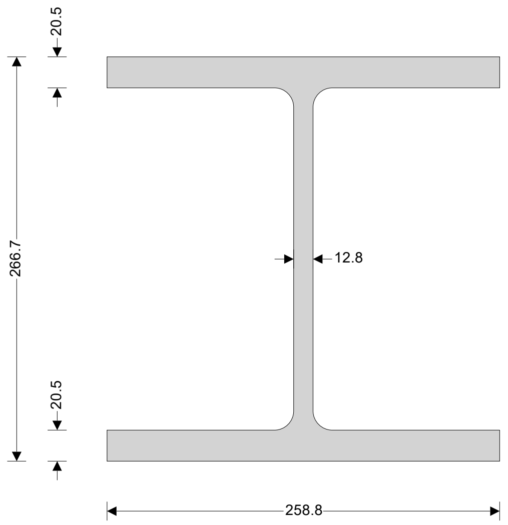
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = \max(t_f, t_w)$ = **20.5** mm

Nominal yield strength f_y = **345** N/mm^2

Nominal ultimate tensile strength f_u = **470** 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 plus midspan

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

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **0.83**

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

Width of section — $c = d$ = **200.3** mm

$c / t_w = 19.0 \times \varepsilon <= 72 \times \varepsilon$ — Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section — $c = (b - t_w - 2 \times r) / 2$ = **110.3** mm

$c / t_f = 6.5 \times \varepsilon <= 9 \times \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 \times t_f$ = **225.7** mm

Shear area factor — η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **81.3** kN

Shear area - cl 6.2.6(3) — $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **3811** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **759** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 1 segment 1 major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_seg1\_max}), \text{abs}(M_{s1\_seg1\_min}))$ = **188.3** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **512.1** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 — k_c = **0.81**

$C_1 = 1 / k_c^2$ = **1.525**

Curvature factor — $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.813**

Poissons ratio — ν = **0.3**

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

Unrestrained length — $L = 1.0 \times L_{s1\_seg1}$ = **2800** mm

Elastic critical buckling moment — $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **4556.4** kNm

Slenderness ratio for lateral torsional buckling — $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.335**

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

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

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection — $\delta_{lim} = L_{s1} / 250$ = **22.4** mm

Maximum deflection span 1 — $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **11.008** mm

PASS - Maximum deflection does not exceed deflection limit