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STRUCTURAL DESIGN SHEETS

(CALCULATIONS)

Structural Engineer | Aleur Oly Rahman

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Date	Sheet No.
Eng. AOR	I.01
Job No.	

Design Data & Characteristic Actions & Loadings.

⊙ Unit Weights (density) of basic Construction materials. kN/m^3

Steel	-	78.5
Aluminium	-	24.0
Concrete	-	24.0 (25.0 RC)
Brick	-	22.5
Timber	-	6.0

⊙ Load kN/m^2 per mm thickness.

Brick wall - 0.021 kN/m^2 per mm
Block wall - 0.012 kN/m^2 per mm

* Unless otherwise stated, all references are to EN 1993-1-1: 2005.

⊙ Partial factors for actions, ULS

Partial factor for permanent actions, $\gamma_g = 1.35$
Partial factor for variable actions, $\gamma_q = 1.50$
Reduction factor, $\xi = 0.85$

⊙ Characteristic actions

	g_k (kN/m^2)	q_k (kN/m^2)
Timber floorboards, Plywood 0.6kg/m³ (22mm)	0.13	—
Rigid insulation, 25mm 7.32 kg/m²· (75mm)	0.22	—
Timber Joists, (max) $=(0.200\times0.050\times1.0)\times\frac{1.1}{0.4}\times4.2$	0.10	—
Plasterboard, gypsum & skim	0.15	—
, Services (ceiling)	0.30	—
Timber floor, (Residential floors)	0.6	1.5
, (Residential balcones)	0.6	2.5
, (Office, general)	0.6	2.5
, (Office, filling)	0.6	5.0
Rafters, battens & roofing felt	0.14	—
Slate	0.60	—
Flat, timber roof	0.75	0.75
Sloped, timber roof	1.25	0.75
Stud Partition wall	0.50	—
Solid b'wk wall, t = 225mm	5.10	—
, t = 103mm	2.50	—
Cavity wall, t_e = 200mm, t_c = 100mm	4.10	—
Glass (19mm max), float	0.49	—
Screed, 25mm (sand/cement)	0.60	—

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Date 07/11/14	Sheet No.
Eng. AoR	I02
Job No. 14.647	

Epworth, Antrim Road
London NW3
↳ Characteristic Actions & design loadings

• T1 – Span = 2400mm

(flat roof load) $g_{kfr} = 0.75 \times 2.7/2 = 1.0\,kN/m$
$q_{kfr} = 0.75 \times 2.7/2 = 1.0\,kN/m$

2No. 50x150 ∴ (C16)
$V_{max} = 3.5\,kN$

• T2 – Span = 3500mm

$P_{T1} = \frac{3.5}{1.4} = 2.5\,kN$ @ 700mm
@ 2700mm.

(flat roof load) $g_{kfr} = 0.75 \times 0.4 = 0.3\,kN/m$
$q_{kfr} = 0.75 \times 0.4 = 0.3\,kN/m$

3No. 50x150 ∴ (C24)
$V_{max} = 5.2\,kN$

• B1 – Span = 4100mm

(Solid 225mm b'wk wall) $g_{kw} = 5.1 \times 1.5 = 7.65\,kN/m.$

(Stud wall) $g_{ksw} = 1.0 \times 3 = 3\,kN/m$

(floor Joists) $g_{kf} = 0.6 \times 3/2 = 0.9\,kN/m.$
$q_{kf} = 1.5 \times 3/2 = 2.25\,kN/m.$

(Sloped Roof) $g_{kR} = 1.25 \times 3/2 = 1.9\,kN/m.$
$q_{kR} = 0.75 \times 3/2 = 1.2\,kN/m.$

$P_{T2} = \frac{5.2}{1.4} = 3.71\,kN$ @ 700mm
@ 3300mm.

∴ 152UC37
$V_{max} = 54\,kN$

• B2 – Span = 3500mm.

(Solid 103mm b'wk wall) $g_{kw} = 2.5 \times 6 = 15\,kN/m$

(floor load x2) $g_{kfl} = 0.6 \times 6.7/2 \times 2 = 4\,kN/m.$
$q_{kfl} = 1.5 \times 6.7/2 \times 2 = 10\,kN/m.$

(Roof load) $g_{kR} = 1.25 \times 2 = 2.5\,kN/m$
$q_{kR} = 0.75 \times 2 = 1.5\,kN/m.$

∴ 203UC46
$V_{max} = 82\,kN$

Padstone // $\frac{82 \times 10^3}{1.5 \times 1.4 \times 200 \times 0.42} = 465$ ∴ 500x200x250 dp (C20)

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Date 07/11/14
Eng. AOR
Job No. 14.647
Sheet No. I03

Epworth, Antrim Road
London NW3
Characteristic Actions & Design Loadings.

• L1 – span = 6500mm

$P_{B1} = \frac{54}{1.4} = 39kN$ @ 3700mm

∴ 203UC46

(Cavity Wall) $g_{kcw} = 4.5 \times 0.7 = 3.2 kN/m$

$V_{max} = 49kN$
$V_{min} = 42kN$

(flat roof load) $g_{kfr} = 0.75 \times 0.4 = 0.3 kN/m$
$q_{kfr} = 0.75 \times 0.4 = 0.3 kN/m$

• C1 – height = 3000mm (max)

$N_{Ed} = 49kN$ ∴ $M_{y,Ed} = 49 \times 0.2 = 10kNm$.

∴ 100×100×6.3 SHS.

Thus Pad foundation ∥ $\sqrt{\frac{49}{1.4 \times 100}} = 0.59m$ ∴ 600SQ × 1100dp Pad Footings.

• B3 – Span = 6400mm

$P_{B1} = \frac{54}{1.4} = 39kN$ @ 3500mm.

$P_{B2} = \frac{82}{1.4} = 59kN$ @ 3500mm.

(Solid 225mm b'wk Wall) $g_{kw} = 5.1 \times 3.1 \times 0.85 = 13.5kN/m$ @ 0 – 3500mm

(flat Roof load) $g_{kfr} = 0.75 \times 0.4 = 0.3 kN/m$
$q_{kfr} = 0.75 \times 0.4 = 0.3 kN/m$ @ 0-3500mm

(floor load) $g_{kfl} = 0.6 \times 0.4 = 0.24 kN/m$
$q_{kfl} = 1.5 \times 0.4 = 0.60 kN/m$ @ 0 – 3500mm

∴ 254×254×89 UC
$V_{max} = 120kN$
$V_{min} = 100kN$
$\delta = 7.9mm$.

$g_{kfl} = 0.6 \times 0.4 \times 2 = 0.48 kN/m$
$q_{kfl} = 1.5 \times 0.4 \times 2 = 1.20 kN/m$ @ 3500 – 6400mm

(Roof load – Sloped) $g_{kfr} = 1.25 \times 0.5 = 0.63 kN/m$
$q_{kfr} = 0.75 \times 0.5 = 0.40 kN/m$. @ 0 – 3500mm

• C2 – height = 3000mm

$N_{Ed} = 120kN$ ∴ $M_{y,Ed} = 120 \times 0.2 = 24kNm$

∴ 152UC30 or 150×100×6.3 RHS.

y
z — z
y

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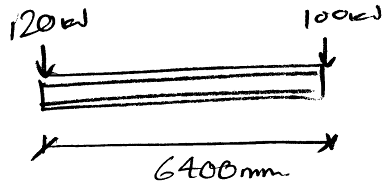
- B3 - Ground beam bearing



$\therefore \frac{(120+100)\,kN}{6.4 \times (1.4)} = UDL = 25\,kN/m$

$\therefore \frac{25}{q_b} = \frac{25}{100} = 0.25$

$\therefore$ 250 wd
× 600
footing reqd.

- (B4) - Span = 1700mm

(Solid 103mm b'wk wall) $g_{kw} = 2.5 \times 6 \times 0.85 = 13\,kN/m$.

(Stud-wall Partition) $g_{ksw} = 0.5 \times 3 = 1.5\,kN/m$

(floor load ×2) $g_{kf} = 0.6 \times 0.8 \times 2 = 1\,kN/m$
$q_{kf} = 1.5 \times 0.8 \times 2 = 2.4\,kN/m$.

$w = 17.9\,kN/m$

$\therefore M_{y,rd} = 6.5\,kNm < 9.64\,kNm$

$\therefore$ 100×140dp P.C.B

$V_{max} = 15\,kN$.

- (B5) - Span = 2600mm

(floor load) $g_{kf} = 0.6 \times 2 = 1.2\,kN/m$
$q_{kf} = 1.5 \times 2 = 3.0\,kN/m$.

Padstone // 150×100×100dp (C20)

$\therefore$ 152×89×16 UB

$V_{max} = 11\,kN$

- (B6) - Span = 1400mm

$P_{B5} = \frac{11}{1.4} = 7.9\,kN$ @ 500mm.

(floor load ×2) $g_{kf} = 0.6 \times \frac{6.5}{2} \times 2 = 3.9\,kN/m$.
$q_{kf} = 1.5 \times \frac{6.5}{2} \times 2 = 10\,kN/m$

(Solid 103mm b'wk wall) $g_{kw} = 2.5 \times 3 = 7.5\,kN/m$.

(Roof load) $g_{kR} = 1.25 \times 1 = 1.25\,kN/m$
$q_{kR} = 0.75 \times 1 = 0.75\,kN/m$

Padstone // 350×100×200dp (C20)

+10mm TOP PLATE
$\therefore$ 152×89×16 UB

$V_{max} = 30\,kN$

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Date 07/11/14	Sheet No. I05
Eng. AOR	
Job No. 14647	

Epworth, Antrim Road.
London NW3
4 Characteristic Actions & Design loadings.

• B8 – Span = 4500mm

$P_{B7} = \frac{26}{1.4} \approx 19kN$ @ 3400mm.

(floor load) $g_{ef} = 0.6 \times 3/2 = 0.9kN/m$
$q_{ef} = 1.5 \times 3/2 = 2.25kN/m$

Padstone # 350x100x200dp (C25)

∴ 152UC30
$V_{max} = 31kN$
$V_{min} = 18kN$.

• B9 – Span = 2000mm (max) – (B4 Similar)

(Solid (0.3m) blk wall) $g_{kw} = 2.5 \times 3 = 7.5kN/m$

(Stud wall.P) $g_{ks} = 0.5 \times 3 = 1.5kN/m$

(floor load) $g_{kf} = 0.6 \times 0.8 = 0.5kN/m$
$q_{kf} = 1.5 \times 0.8 = 1.2kN/m$

$w = 10.7kN/m$

∴ $M_{(ed)} = 5.35kNm < 6.3kNm$ ∴

USE 100x140dp P.C.B.

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Date 15/01/15	Sheet No.
Eng. AOR	I06
Job No. 14.647	

Epsworth, Antrim Road
London NW3
↳ Characteristic Actions & Design Loadings.

Finalised Scheme – B7 (Crank frame) ommitted
+ additional chimney B support
@ Ground floor (Structure above)

• (B8) – Span = 4500mm (revised)

(Chimney Breast) $g_{k\,chim\,B} = 22.5 \times 0.3 \times 5 \times 0.7$ (70%)
≈ 24 kN/m. @ 1200 – 4500 mm.

∴ 203UC46

$V_{min} = 41$kN
$V_{max} = 69$kN.

Padstone // $\frac{67 \times 10^3 \times 2}{1.5 \times 1.4 \times 100 \times 0.42} = 1565$ ∴ 1200 Ly × 100 wd × 215 dp (max)
450 × 100 × 250 dp (min)

• (B9) – Span = 2200mm.

(Chimney Breast) $g_{k\,chim\,B} = 22.5 \times 0.3 \times 8.6 \times 0.7 \approx 41$ kN/m @ 1200-2200mm

∴ 152UC23

$V_{max} = 43$kN.
$V_{min} = 13$kN.

Padstone // 150 × 100 × 100 dp (C20)

• (B5) – Span = 2800mm. (revised)

loading as Previous/Prelim. +
$P_{B9} = \frac{43}{1.4} = 31$kN @ 150mm.

∴ 152×89×16 UB

$V_{max} = 52$kN

• (B10) – Span = 1000mm (max)

$P_{B5,max} = \frac{52}{1.4} = 37$kN @ 800mm.

(Solid b'wk 215mm) $g_{k\,blwk} = 5.1 \times 3.7 = 18.9$ kN/m

(Roof load) $g_{kr} = 1.25 \times 1.5 = 1.9$ kN/m.
$q_{kr} = 0.75 \times 1.5 = 1.2$ kN/m.

(floor load ×2) $g_{kf} = 0.6 \times 1.5 \times 2 = 1.8$ kN/m.
$q_{kf} = 1.5 \times 1.5 \times 2 = 6.0$ kN/m.

∴ 152UC23

$V_{max} = 56$kN

Padstone // 450 × 150 × 250 dp (C20).

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Date 11/05/15	Sheet No.
Eng. AoR	I07
Job No. 14.647	

Epworth, Antrim Road London NW3
↳ Characteristic Actions & Design Loading.

Revised Finalised Scheme - Internal stud-walls (ALL)

- (R1) - Span = 1100mm Rooflight @ Roof level.
  $g_{kR} = 0.75 \times 2/2 = 0.75$ kN/m
  $q_{kR} = 0.75$ kN/m
  ∴ 2No. 50x100 (C16), $V_{max} = 1.2$ kN
- (R2) - Span = 1700mm.
  $P_{R1} = \frac{1.2}{1.4} = 0.9$ kN @ 350mm, @ 1250mm
  ∴ 2No. 50x100 (C16), $V_{max} = 2.1$ kN.
- (C3) - height = 6000mm. Bearing as/for B8.1/2
  $N_{Ed} = 69 \times 2 = 138$ kN ∴ $M_{y,Ed} = 69 \times 0.2 = 14$ kNm (worst case)
  Pad footing (if Req) // $\sqrt{\frac{134}{100 \times 1.4}} = 0.97$ ∴ 1000SQ x 600 dp
  ∴ 120x120x8.0 SHS
- (B4) & (B9) - Span (max) = 2200mm (in stud-wall - L-6)
  (floor load) $g_{kf} = 0.6 \times 4/2 = 1.2$ kN/m
  $q_{kf} = 1.5 \times 4/2$ 3kN/m
  ∴ 2No. 50x200 (C24), $V_{max} = 6.8$ kN
- (P1) - height 2500mm
  $N_{Ed} = 6.8$ kN ∴ $M_{x,Ed} = 6.8 \times 0.1 = 0.68$ kNm
  ∴ 50x200 (C24) Post
- (B2) & (B6) bear onto new columns & foundation.
  (C4) - height = 3000mm
  $N_{Ed} = P_{B2} = 82$ kN ∴ $M_{x,Ed} = 82 \times 0.2 = 17$ kNm
  Pad footing // $\sqrt{\frac{82}{1.4 \times 100}} = 0.77$ ∴ 800SQ x 600 dp (C35)
  ∴ 120x120x6.3 SHS.
  (C5) - height = 3000mm
  $N_{Ed} = P_{B6} = 30$ kN ∴ $M_{x,Ed} = 30 \times 0.2 = 6$ kNm
  Pad footing // $\sqrt{\frac{30}{1.4 \times 100}} = 0.46$ ∴ 500 x 600 dp (C35)
  ∴ 90x90x4.0 SHS.

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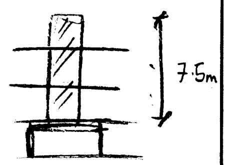
Date 18/05/15	Sheet No.
Eng. AoR	I08
Job No. 14·647	

Epworth, Antrim Road
London NW3
↳ Characteristic Actions & Design loadings.

• (B11) – Span = 3600mm.

(Chimney breast load) $g_{k,cb} = 22.5 \times 0.35 \times 7.5 \times 0.7$
$= 25\,kN/m$ @ 1400–3600



Elevation.

∴ 152UC30

(floor load) $g_{kf} = 0.6 \times 3/2 = 0.9\,kN/m$
$q_{kf} = 1.5 \times 3/2 = 2.25\,kN/m$.

$V_{max} = 53\,kN$
$V_{min} = 24\,kN$

• (C6) – height = 3000mm

$N_{Ed} = 24\,kN$ ∴ $M_{y,nod} = 24 \times 0.2 = 4.8\,kNm$

∴ 100×100×4.0 SHS.

Pad footing if Req. // $\sqrt{\frac{24}{100}} = 0.49$ ∴ 600sq × 600dp (C35)

∴ B3 Revised & Check. → Exst. <u>ok</u>

∴ C2 Revised & Check. → Exst. <u>ok</u>

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Date 06/07/15	Sheet No. I09
Eng. AOR	
Job No. 14.647	

Epworth, Antrim Road
London NW3
Characteristic Actions & Design Loadings.

- (T3) - Span = 1500mm

(Floor loading) $g_{kf} = 0.6 \times 2.5/2 = 0.75$ kN/m
$q_{kf} = 1.5 \times 2.5/2 = 1.90$ kN/m

(Stud-wall) $g_{ksw} = 0.5 \times 2 = 1$ kN/m

∴ 1No. 50x200 (C16) Min. Req.
$V_{max} = 3.7$ kN.

- (T4) - Span = 3000mm

(Floor loading) $g_{kf} = 0.6 \times 0.4 = 0.24$ kN/m
$q_{kf} = 1.5 \times 0.4 = 0.60$ kN/m

(Stud-wall) $g_{ksw} = 0.5 \times 2 = 1$ kN/m @ 0-1000mm

(Roof load) $\left.\begin{array}{l} g_{kel} = 1.25 \times 1/2 = 0.63 \text{ kN/m} \\ q_{kel} = 0.75 \times 1/2 = 0.38 \text{ kN/m} \end{array}\right\}$ @ 0-1000mm

$P_{T3} = \frac{3.9}{1.4} = 2.79$ kN @ 500mm

∴ 2No. 50x200 (C16)
$V_{max} = 7.4$ kN.

- (B11) - Additional Point Load - Design Check

$P_{T4} = \frac{7.4}{1.4} = 5.3$ kN @ 1200mm.

∴ 152UC30 Adequate

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 1.5.08

Analysis results

Design moment M = **2.096** kNm — Design shear F = **3.494** kN

Total load on member W_{tot} = **6.988** kN

Reactions at support A R_{A_max} = **3.494** kN — R_{A_min} = **3.494** kN

Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **1.255** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **1.200** kN

Reactions at support B R_{B_max} = **3.494** kN — R_{B_min} = **3.494** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **1.255** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **1.200** kN

Timber section details

Breadth of section b = **50** mm — Depth of section h = **150** mm

Number of sections N = **2** — Breadth of member b_b = **100** mm

Timber strength class **C16**

Member details

Service class of timber **1** — Load duration **Permanent**

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.349** N/mm^2 — Design compressive strength $f_{c.90.d}$ = **1.015** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **5.590** N/mm^2 — Design bending strength $f_{m.d}$ = **7.385** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.521** N/mm^2 — Permissible shear stress $f_{v.d}$ = **1.477** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **9.600** mm — Total final deflection δ_{fin} = **5.806** mm

PASS - Total final deflection is less than the 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 3.0.08

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = **7.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **10.9** kN	V_{min} = **-10.9** kN
Deflection	δ_{max} = **1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **10.9** kN	R_{A_min} = **10.9** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **3.7** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **3.9** kN	
Maximum reaction at support B	R_{B_max} = **10.9** kN	R_{B_min} = **10.9** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **3.7** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **3.9** kN	

Section details

Section type **UB 152x89x16 (BS4-1)** Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = **11** kN Design shear resistance $V_{c,Rd}$ = **129.8** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **7.1** kNm Des.bending resist.moment $M_{c,Rd}$ = **33.9** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **0.906** Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **26.4** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection δ_{lim} = **7.2** mm Maximum deflection δ = **1.019** 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 3.0.08

Support conditions

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

Analysis results

Maximum moment	M_{max} = **10.9** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **10.9** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **10.6** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum shear	V_{max} = **29.6** kN	V_{min} = **-26.6** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **29.6** kN	$V_{s1\_seg1\_min}$ = **-3.8** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **0** kN	$V_{s1\_seg2\_min}$ = **-26.6** kN
Deflection segment 3	δ_{max} = **0.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **29.6** kN	R_{A_min} = **29.6** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **18.3** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **3.3** kN	
Maximum reaction at support B	R_{B_max} = **26.6** kN	R_{B_min} = **26.6** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **16.1** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **3.3** kN	

Section details

Section type	**UB 152x89x16 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force V_{Ed} = **30** kN — Design shear resistance $V_{c,Rd}$ = **129.8** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **10.9** kNm — Des.bending resist.moment $M_{c,Rd}$ = **33.9** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **0.541** — Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **32** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection δ_{lim} = **3.9** mm — Maximum deflection δ = **0.133** 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 3.0.08

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = **18.4** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **32** kN	V_{min} = **-32** kN
Deflection	δ_{max} = **0** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **32** kN	R_{A_min} = **32** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **23.7** kN	
Maximum reaction at support B	R_{B_max} = **32** kN	R_{B_min} = **32** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **23.7** kN	

Section details

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

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = **32** kN Design shear resistance $V_{c,Rd}$ = **156.4** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **18.4** kNm Des.bending resist.moment $M_{c,Rd}$ = **47.1** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **1.262** Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **25.6** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection δ_{lim} = **6.4** mm Maximum deflection δ = **0** 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 3.0.08

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = **30.6** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **17.5** kN	V_{min} = **-30.6** kN
Deflection	δ_{max} = **3.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **17.5** kN	R_{A_min} = **17.5** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **7.3** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **5.1** kN	
Maximum reaction at support B	R_{B_max} = **30.6** kN	R_{B_min} = **30.6** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **17** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **5.1** kN	

Section details

Section type	**UC 152x152x30 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force V_{Ed} = **31** kN   Design shear resistance $V_{c,Rd}$ = **183.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **30.6** kNm   Des.bending resist.moment $M_{c,Rd}$ = **68.1** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **0.779**   Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **58.2** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection δ_{lim} = **12.5** mm   Maximum deflection δ = **3.273** 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 3.0.08

Support conditions

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

Analysis results

Maximum moment	M_{max} = **72.3** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **40.6** kN	V_{min} = **-69.1** kN
Deflection	δ_{max} = **11.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **40.6** kN	R_{A_min} = **40.6** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **30.1** kN	
Maximum reaction at support B	R_{B_max} = **69.1** kN	R_{B_min} = **69.1** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **51.2** kN	

Section details

Section type	**UC 203x203x46 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force	V_{Ed} = **69** kN	Design shear resistance	$V_{c,Rd}$ = **269.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	M_{Ed} = **72.3** kNm	Des.bending resist.moment	$M_{c,Rd}$ = **136.8** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio	$\bar{\lambda}_{LT}$ = **1.050**	Limiting slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **91.5** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection	δ_{lim} = **18** mm	Maximum deflection	δ = **11.446** 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 3.0.08

Support conditions

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

Analysis results

Maximum moment	M_{max} = **16.7** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **12.9** kN	V_{min} = **-43.1** kN
Deflection	δ_{max} = **2.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **12.9** kN	R_{A_min} = **12.9** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **9.6** kN	
Maximum reaction at support B	R_{B_max} = **43.1** kN	R_{B_min} = **43.1** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **31.9** kN	

Section details

Section type	**UC 152x152x23 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 3**		

Check shear - Section 6.2.6

Design shear force V_{Ed} = **43** kN Design shear resistance $V_{c,Rd}$ = **129.8** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **16.7** kNm Des.bending resist.moment $M_{c,Rd}$ = **45.1** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **0.752** Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **38** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = **8.8** mm Maximum deflection δ = **2.085** 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 3.0.08

Support conditions

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

Analysis results

Maximum moment	M_{max} = **11.6** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **51.3** kN	V_{min} = **-13.9** kN
Deflection	δ_{max} = **4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **51.3** kN	R_{A_min} = **51.3** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **33.3** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **4.2** kN	
Maximum reaction at support B	R_{B_max} = **13.9** kN	R_{B_min} = **13.9** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **5.7** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **4.2** kN	

Section details

Section type	**UB 152x89x16 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force V_{Ed} = **51** kN   Design shear resistance $V_{c,Rd}$ = **129.8** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **11.6** kNm   Des.bending resist.moment $M_{c,Rd}$ = **33.9** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **1.518**   Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **14.2** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = **11.2** mm   Maximum deflection δ = **3.987** 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 3.0.08

Support conditions

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

Analysis results

Maximum moment	M_{max} = **14.9** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **14.9** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **12.7** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum shear	V_{max} = **55.8** kN	V_{min} = **-35.8** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **55.8** kN	$V_{s1\_seg1\_min}$ = **-15** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **0** kN	$V_{s1\_seg2\_min}$ = **-35.8** kN
Deflection segment 3	δ_{max} = **0.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **55.8** kN	R_{A_min} = **55.8** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **37.3** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **3.6** kN	
Maximum reaction at support B	R_{B_max} = **35.8** kN	R_{B_min} = **35.8** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **22.5** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **3.6** kN	

Section details

Section type	**UC 152x152x23 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 3**		

Check shear - Section 6.2.6

Design shear force	V_{Ed} = **56** kN	Design shear resistance	$V_{c,Rd}$ = **129.8** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	M_{Ed} = **14.9** kNm	Des.bending resist.moment	$M_{c,Rd}$ = **45.1** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio	$\bar{\lambda}_{LT}$ = **0.204**	Limiting slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.400**

$\bar{\lambda}_{LT}$ < $\bar{\lambda}_{LT,0}$ - 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	δ_{lim} = **4** mm	Maximum deflection	δ = **0.384** mm

PASS - Maximum deflection does not exceed deflection limit

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 1.5.08

Analysis results

Design moment M = **0.330** kNm — Design shear F = **1.198** kN

Total load on member W_{tot} = **2.396** kN

Reactions at support A R_{A_max} = **1.198** kN — R_{A_min} = **1.198** kN

Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **0.429** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **0.413** kN

Reactions at support B R_{B_max} = **1.198** kN — R_{B_min} = **1.198** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **0.429** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **0.413** kN

Timber section details

Breadth of section b = **50** mm — Depth of section h = **100** mm

Number of sections N = **2** — Breadth of member b_b = **100** mm

Timber strength class **C16**

Member details

Service class of timber **1** — Load duration **Permanent**

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.120** N/mm^2 — Design compressive strength $f_{c.90.d}$ = **1.015** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **1.977** N/mm^2 — Design bending strength $f_{m.d}$ = **8.008** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.268** N/mm^2 — Permissible shear stress $f_{v.d}$ = **1.477** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **4.400** mm — Total final deflection δ_{fin} = **0.688** mm

PASS - Total final deflection is less than the deflection limit

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 1.5.08

Analysis results

Design moment M = **4.005** kNm; Design shear F = **5.150** kN

Total load on member W_{tot} = **10.107** kN

Reactions at support A R_{A_max} = **5.150** kN; R_{A_min} = **5.150** kN

Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **3.232** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **0.525** kN

Reactions at support B R_{B_max} = **4.957** kN; R_{B_min} = **4.957** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **3.089** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **0.525** kN

Timber section details

Breadth of section b = **50** mm; Depth of section h = **150** mm

Number of sections N = **3**; Breadth of member b_b = **150** mm

Timber strength class **C24**

Member details

Service class of timber **1**; Load duration **Permanent**

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.343** N/mm²; Design compressive strength $f_{c.90.d}$ = **1.154** N/mm²

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **7.120** N/mm²; Design bending strength $f_{m.d}$ = **11.077** N/mm²

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.512** N/mm²; Permissible shear stress $f_{v.d}$ = **1.846** N/mm²

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **14.000** mm; Total final deflection δ_{fin} = **13.663** mm

PASS - Total final deflection is less than the deflection limit

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 1.5.08

Analysis results

Design moment M = **0.813** kNm — Design shear F = **2.048** kN

Total load on member W_{tot} = **3.953** kN

Reactions at support A R_{A_max} = **2.048** kN — R_{A_min} = **2.048** kN

Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **1.234** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **0.255** kN

Reactions at support B R_{B_max} = **1.905** kN — R_{B_min} = **1.905** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **1.128** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **0.255** kN

Timber section details

Breadth of section b = **50** mm — Depth of section h = **100** mm

Number of sections N = **2** — Breadth of member b_b = **100** mm

Timber strength class **C16**

Member details

Service class of timber **1** — Load duration **Permanent**

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.205** N/mm² — Design compressive strength $f_{c.90.d}$ = **1.015** N/mm²

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **4.875** N/mm² — Design bending strength $f_{m.d}$ = **8.008** N/mm²

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.459** N/mm² — Permissible shear stress $f_{v.d}$ = **1.477** N/mm²

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

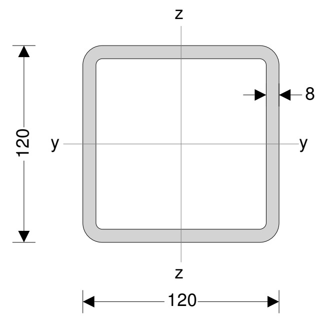
Deflection limit δ_{lim} = **6.800** mm — Total final deflection δ_{fin} = **4.536** mm

PASS - Total final deflection is less than the deflection limit

STEEL COLUMN DESIGN (EN 1993-1-1)

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

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section **SHS 120x120x8.0**

System length y axis buckling L_y = **6000** mm   System length z axis buckling L_z = **6000** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load N_{Ed} = **138** kN (Compression)

Moment about y axis at end 1 $M_{y,Ed1}$ = **14.0** kNm   Moment about y axis at end 2 $M_{y,Ed2}$ = **14.0** kNm

Single curvature bending about y axis

Moment about z axis at end 1 $M_{z,Ed1}$ = **1.0** kNm   Moment about z axis at end 2 $M_{z,Ed2}$ = **1.0** kNm

Single curvature bending about z axis

Shear force parallel to z axis $V_{z,Ed}$ = **1** kN   Shear force parallel to y axis $V_{y,Ed}$ = **1** kN

Material details

Steel grade **S275**

Yield strength f_y = **275** N/mm²   Ultimate strength f_u = **410** N/mm²

Modulus of elasticity E = **210** kN/mm²   Shear modulus G = **80.8** kN/mm²

Buckling length for flexural buckling about y axis

End restraint factor K_y = **1.000**   Buckling length L_{cr_y} = **6000** mm

Buckling length for flexural buckling about z axis

End restraint factor K_z = **1.000**   Buckling length L_{cr_z} = **6000** mm

Section classification (Table 5.2)

Web classification **1**   Flange classification **1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force $V_{z,Ed}$ = **1.0** kN   Plastic shear resistance $V_{pl,z,Rd}$ = **279.1** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

$V_{z,Ed}$ <= 0.5×$V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = **1.0** kN — Plastic shear resistance $V_{pl,y,Rd}$ = **279.1** kN

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed}$ <= 0.5×$V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = **138** kN — Design resistance $N_{c,Rd}$ = **967** kN

PASS - The compression design resistance exceeds the design force

Bending about y axis (cl. 6.2.5)

Design bending moment $M_{y,Ed}$ = **14.0** kNm — Design resistance $M_{c,y,Rd}$ = **40.3** kNm

PASS - The bending design resistance about the y axis exceeds the design moment

Bending about z axis (cl. 6.2.5)

Design bending moment $M_{z,Ed}$ = **1.0** kNm — Design resistance $M_{c,z,Rd}$ = **40.3** kNm

PASS - The bending design resistance about the z axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about y axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed}$ = **14.0** kNm — Modified design resistance $M_{N,y,Rd}$ = **40.3** kNm

PASS - Bending resistance about y axis in presence of axial load exceeds design moment

Bending about z axis (cl. 6.2.9.1)

Design bending moment $M_{z,Ed}$ = **1.0** kNm — Modified design resistance $M_{N,z,Rd}$ = **40.3** kNm

PASS - Bending resistance about z axis in presence of axial load exceeds design moment

Biaxial bending

Section utilisation at end 1 UR_{CS_1} = **0.168** — Section utilisation at end 2 UR_{CS_2} = **0.168**

PASS - The cross-section resistance is adequate

Buckling resistance (cl. 6.3)

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = **351.8** kN — Flexural buck resist about z $N_{b,z,Rd}$ = **351.8** kN

Min. buckling resistance $N_{b,Rd}$ = **351.8** kN

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Design bending moment $M_{y,Ed}$ = **14.0** kNm

Lat. torsional buck length fact K_{LT} = **1.00** — Design buckling resistance mt $M_{b,Rd}$ = **40.3** kNm

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

Section utilisation UR_{B_1} = **0.869** — Section utilisation UR_{B_2} = **0.699**

PASS - The buckling resistance is adequate

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 1.5.08

Analysis results

Design moment M = **3.759** kNm | Design shear F = **6.834** kN

Total load on member W_{tot} = **13.668** kN

Reactions at support A R_{A_max} = **6.834** kN | R_{A_min} = **6.834** kN

Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **1.396** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **3.300** kN

Reactions at support B R_{B_max} = **6.834** kN | R_{B_min} = **6.834** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **1.396** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **3.300** kN

Timber section details

Breadth of section b = **50** mm | Depth of section h = **200** mm

Number of sections N = **2** | Breadth of member b_b = **100** mm

Timber strength class **C24**

Member details

Service class of timber **1** | Load duration **Permanent**

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.683** N/mm^2 | Design compressive strength $f_{c.90.d}$ = **1.154** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **5.638** N/mm^2 | Design bending strength $f_{m.d}$ = **11.077** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.765** N/mm^2 | Permissible shear stress $f_{v.d}$ = **1.846** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **8.800** mm | Total final deflection δ_{fin} = **2.610** mm

PASS - Total final deflection is less than the deflection limit

Timber member designTimber member designTimber member designTIMBER MEMBER DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 1.5.08

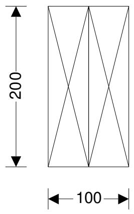
Analysis results

Design moment in major axis M_y = **1.400** kNm

Design moment in minor axis M_z = **1.400** kNm

Design shear F = **1.000** kN

Maximum reaction R = **6.800** kN



Timber section details

Breadth of timber sections b = **50** mm

Depth of timber sections h = **200** mm

Number of timber sections in member N = **2**

Overall breadth of timber member $b_b = N \times b$ = **100** mm

Timber strength class - EN 338:2009 Table 1 **C24**

Member details

Load duration - cl.2.3.1.2 **Permanent**

Service class of timber - cl.2.3.1.3 **1**

Length of bearing L_b = **100** mm

Section properties

Cross sectional area of member $A = N \times b \times h$ = **20000** mm^2

Section modulus $W_y = N \times b \times h^2 / 6$ = **666667** mm^3

$W_z = h \times (N \times b)^2 / 6$ = **333333** mm^3

Second moment of area $I_y = N \times b \times h^3 / 12$ = **66666667** mm^4

$I_z = h \times (N \times b)^3 / 12$ = **16666667** mm^4

Radius of gyration $r_y = \sqrt{(I_y / A)}$ = **57.7** mm

$r_z = \sqrt{(I_z / A)}$ = **28.9** mm

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 γ_M = **1.300**

Modification factors

Modification factor for load duration and moisture content - Table 3.1

k_{mod} = **0.600**

Deformation factor for service classes - Table 3.2 k_{def} = **0.600**

Depth factor for bending - exp.3.1 $k_{h.m}$ = **1.000**

Depth factor for tension - exp.3.1 $k_{h.t}$ = **1.000**

Bending stress re-distribution factor - cl.6.1.6(2) k_m = **0.700**

Crack factor for shear resistance - cl.6.1.7(2) k_{cr} = **0.670**

Load configuration factor - exp.6.4 $k_{c.90}$ = **1.000**

System strength factor - cl.6.6 k_{sys} = **1.000**

Effective length - Table 6.1 $L_{ef} = 1.0 \times L_s$ = **4000** mm

Critical bending stress - exp.6.32 $\sigma_{m.crit} = 0.78 \times (N \times b)^2 \times E_{0.05} / (h \times L_{ef})$ = **72.150** N/mm²

Relative slenderness for bending - exp.6.30 $\lambda_{rel.m} = \sqrt{[f_{m.k} / \sigma_{m.crit}]}$ = **0.577**

Lateral buckling factor - exp.6.34 k_{crit} = **1.000**

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress $\sigma_{c.90.d} = R / (N \times b \times L_b)$ = **0.680** N/mm²

Design compressive strength $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M$ = **1.154** N/mm²

$\sigma_{c.90.d} / f_{c.90.d}$ = **0.589**

PASS - Design compressive strength exceeds design compressive stress at bearing

Biaxial bending - cl 6.1.6

Design bending stress in major (y-y) axis $\sigma_{m.y.d} = M_y / W_y$ = **2.100** N/mm²

Design bending stress in minor (z-z) axis $\sigma_{m.z.d} = M_z / W_z$ = **4.200** N/mm²

Design bending strength $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M$ = **11.077** N/mm²

Combined bending checks - eq.6.11 & eq.6.12 $\sigma_{m.y.d} / f_{m.d} + k_m \times \sigma_{m.z.d} / f_{m.d}$ = **0.455**

$k_m \times \sigma_{m.y.d} / f_{m.d} + \sigma_{m.z.d} / f_{m.d}$ = **0.512**

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 3 \times F / (2 \times k_{cr} \times A)$ = **0.112** N/mm²

Permissible shear stress $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M$ = **1.846** N/mm²

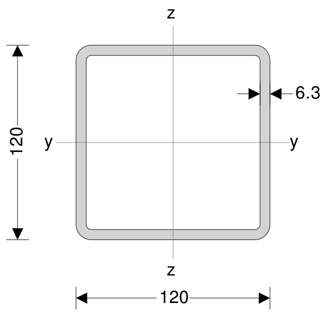
$\tau_d / f_{v.d}$ = **0.061**

PASS - Design shear strength exceeds design shear stress

STEEL COLUMN DESIGN (EN 1993-1-1)

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

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section **SHS 120x120x6.3**

System length y axis buckling L_y = **3000** mm System length z axis buckling L_z = **3000** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load N_{Ed} = **82** kN (Compression)

Moment about y axis at end 1 $M_{y,Ed1}$ = **17.0** kNm Moment about y axis at end 2 $M_{y,Ed2}$ = **17.0** kNm

Single curvature bending about y axis

Moment about z axis at end 1 $M_{z,Ed1}$ = **1.0** kNm Moment about z axis at end 2 $M_{z,Ed2}$ = **1.0** kNm

Single curvature bending about z axis

Shear force parallel to z axis $V_{z,Ed}$ = **1** kN Shear force parallel to y axis $V_{y,Ed}$ = **1** kN

Material details

Steel grade **S275**

Yield strength f_y = **275** N/mm² Ultimate strength f_u = **410** N/mm²

Modulus of elasticity E = **210** kN/mm² Shear modulus G = **80.8** kN/mm²

Buckling length for flexural buckling about y axis

End restraint factor K_y = **1.000** Buckling length L_{cr_y} = **3000** mm

Buckling length for flexural buckling about z axis

End restraint factor K_z = **1.000** Buckling length L_{cr_z} = **3000** mm

Section classification (Table 5.2)

Web classification **1** Flange classification **1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force $V_{z,Ed}$ = **1.0** kN Plastic shear resistance $V_{pl,z,Rd}$ = **224.1** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

$V_{z,Ed}$ <= 0.5×$V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = **1.0** kN Plastic shear resistance $V_{pl,y,Rd}$ = **224.1** kN

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed}$ <= 0.5×$V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = **82** kN Design resistance $N_{c,Rd}$ = **776** kN

PASS - The compression design resistance exceeds the design force

Bending about y axis (cl. 6.2.5)

Design bending moment $M_{y,Ed}$ = **17.0** kNm Design resistance $M_{c,y,Rd}$ = **32.9** kNm

PASS - The bending design resistance about the y axis exceeds the design moment

Bending about z axis (cl. 6.2.5)

Design bending moment $M_{z,Ed}$ = **1.0** kNm Design resistance $M_{c,z,Rd}$ = **32.9** kNm

PASS - The bending design resistance about the z axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about y axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed}$ = **17.0** kNm Modified design resistance $M_{N,y,Rd}$ = **32.9** kNm

PASS - Bending resistance about y axis in presence of axial load exceeds design moment

Bending about z axis (cl. 6.2.9.1)

Design bending moment $M_{z,Ed}$ = **1.0** kNm Modified design resistance $M_{N,z,Rd}$ = **32.9** kNm

PASS - Bending resistance about z axis in presence of axial load exceeds design moment

Biaxial bending

Section utilisation at end 1 UR_{CS_1} = **0.332** Section utilisation at end 2 UR_{CS_2} = **0.332**

PASS - The cross-section resistance is adequate

Buckling resistance (cl. 6.3)

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = **639.8** kN Flexural buck resist about z $N_{b,z,Rd}$ = **639.8** kN

Min. buckling resistance $N_{b,Rd}$ = **639.8** kN

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Design bending moment $M_{y,Ed}$ = **17.0** kNm

Lat. torsional buck length fact K_{LT} = **1.00** Design buckling resistance mt $M_{b,Rd}$ = **32.9** kNm

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

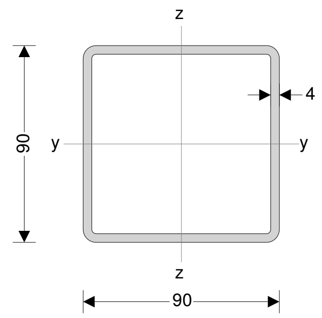
Section utilisation UR_{B_1} = **0.701** Section utilisation UR_{B_2} = **0.493**

PASS - The buckling resistance is adequate

STEEL COLUMN DESIGN (EN 1993-1-1)

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

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section **SHS 90x90x4.0**

System length y axis buckling L_y = **3000** mm   System length z axis buckling L_z = **3000** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load N_{Ed} = **30** kN (Compression)

Moment about y axis at end 1 $M_{y,Ed1}$ = **6.0** kNm   Moment about y axis at end 2 $M_{y,Ed2}$ = **6.0** kNm

Single curvature bending about y axis

Moment about z axis at end 1 $M_{z,Ed1}$ = **1.0** kNm   Moment about z axis at end 2 $M_{z,Ed2}$ = **1.0** kNm

Single curvature bending about z axis

Shear force parallel to z axis $V_{z,Ed}$ = **1** kN   Shear force parallel to y axis $V_{y,Ed}$ = **1** kN

Material details

Steel grade **S275**

Yield strength f_y = **275** N/mm^2 Ultimate strength f_u = **410** N/mm^2

Modulus of elasticity E = **210** kN/mm^2 Shear modulus G = **80.8** kN/mm^2

Buckling length for flexural buckling about y axis

End restraint factor K_y = **1.000**   Buckling length L_{cr_y} = **3000** mm

Buckling length for flexural buckling about z axis

End restraint factor K_z = **1.000**   Buckling length L_{cr_z} = **3000** mm

Section classification (Table 5.2)

Web classification **1**   Flange classification **1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force $V_{z,Ed}$ = **1.0** kN   Plastic shear resistance $V_{pl,z,Rd}$ = **107.9** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

$V_{z,Ed}$ <= 0.5×$V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = **1.0** kN   Plastic shear resistance $V_{pl,y,Rd}$ = **107.9** kN

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed}$ <= 0.5×$V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = **30** kN   Design resistance $N_{c,Rd}$ = **374** kN

PASS - The compression design resistance exceeds the design force

Bending about y axis (cl. 6.2.5)

Design bending moment $M_{y,Ed}$ = **6.0** kNm   Design resistance $M_{c,y,Rd}$ = **12.0** kNm

PASS - The bending design resistance about the y axis exceeds the design moment

Bending about z axis (cl. 6.2.5)

Design bending moment $M_{z,Ed}$ = **1.0** kNm   Design resistance $M_{c,z,Rd}$ = **12.0** kNm

PASS - The bending design resistance about the z axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about y axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed}$ = **6.0** kNm   Modified design resistance $M_{N,y,Rd}$ = **12.0** kNm

PASS - Bending resistance about y axis in presence of axial load exceeds design moment

Bending about z axis (cl. 6.2.9.1)

Design bending moment $M_{z,Ed}$ = **1.0** kNm   Modified design resistance $M_{N,z,Rd}$ = **12.0** kNm

PASS - Bending resistance about z axis in presence of axial load exceeds design moment

Biaxial bending

Section utilisation at end 1 UR_{CS_1} = **0.330**   Section utilisation at end 2 UR_{CS_2} = **0.330**

PASS - The cross-section resistance is adequate

<u>Buckling resistance (cl. 6.3)</u>

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = **251.9** kN   Flexural buck resist about z $N_{b,z,Rd}$ = **251.9** kN

Min. buckling resistance $N_{b,Rd}$ = **251.9** kN

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Design bending moment $M_{y,Ed}$ = **6.0** kNm

Lat. torsional buck length fact K_{LT} = **1.00**   Design buckling resistance mt $M_{b,Rd}$ = **12.0** kNm

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

Section utilisation UR_{B_1} = **0.721**   Section utilisation UR_{B_2} = **0.539**

PASS - The buckling resistance is adequate

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 3.0.08

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = **53.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **54** kN	V_{min} = **-53.7** kN
Deflection	δ_{max} = **2.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **54** kN	R_{A_min} = **54** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **32.1** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **7.1** kN	
Maximum reaction at support B	R_{B_max} = **53.7** kN	R_{B_min} = **53.7** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **31.9** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **7.1** kN	

Section details

Section type	**UC 152x152x37**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force V_{Ed} = **54** kN — Design shear resistance $V_{c,Rd}$ = **226.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **53.8** kNm — Des.bending resist.moment $M_{c,Rd}$ = **84.9** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection δ_{lim} = **11.4** mm — Maximum deflection δ = **2.735** 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 3.0.08

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = **40** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **23.4** kN	V_{min} = **-52.3** kN
Deflection	δ_{max} = **10.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **23.4** kN	R_{A_min} = **23.4** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **17.3** kN	
Maximum reaction at support B	R_{B_max} = **52.3** kN	R_{B_min} = **52.3** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **38.7** kN	

Section details

Section type **UC 152x152x30 (BS4-1)** Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = **52** kN Design shear resistance $V_{c,Rd}$ = **183.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **40** kNm Des.bending resist.moment $M_{c,Rd}$ = **68.1** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **1.073** Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **44.6** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

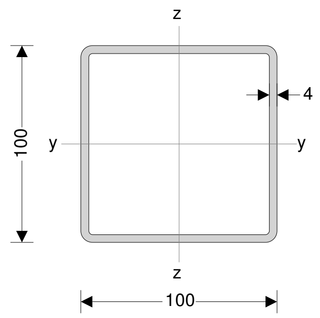
Limiting deflection δ_{lim} = **14.4** mm Maximum deflection δ = **10.24** mm

PASS - Maximum deflection does not exceed deflection limit

STEEL COLUMN DESIGN (EN 1993-1-1)

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

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section **SHS 100x100x4.0**

System length y axis buckling L_y = **6000** mm System length z axis buckling L_z = **6000** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load N_{Ed} = **24** kN (Compression)

Moment about y axis at end 1 $M_{y,Ed1}$ = **5.0** kNm Moment about y axis at end 2 $M_{y,Ed2}$ = **5.0** kNm

Single curvature bending about y axis

Moment about z axis at end 1 $M_{z,Ed1}$ = **1.0** kNm Moment about z axis at end 2 $M_{z,Ed2}$ = **1.0** kNm

Single curvature bending about z axis

Shear force parallel to z axis $V_{z,Ed}$ = **1** kN Shear force parallel to y axis $V_{y,Ed}$ = **1** kN

Material details

Steel grade **S275**

Yield strength f_y = **275** N/mm^2 Ultimate strength f_u = **410** N/mm^2

Modulus of elasticity E = **210** kN/mm^2 Shear modulus G = **80.8** kN/mm^2

Buckling length for flexural buckling about y axis

End restraint factor K_y = **1.000** Buckling length L_{cr_y} = **6000** mm

Buckling length for flexural buckling about z axis

End restraint factor K_z = **1.000** Buckling length L_{cr_z} = **6000** mm

Section classification (Table 5.2)

Web classification **1** Flange classification **1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force $V_{z,Ed}$ = **1.0** kN Plastic shear resistance $V_{pl,z,Rd}$ = **120.6** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

$V_{z,Ed}$ <= $0.5 \times V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = **1.0** kN — Plastic shear resistance $V_{pl,y,Rd}$ = **120.6** kN

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed}$ <= $0.5 \times V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = **24** kN — Design resistance $N_{c,Rd}$ = **418** kN

PASS - The compression design resistance exceeds the design force

Bending about y axis (cl. 6.2.5)

Design bending moment $M_{y,Ed}$ = **5.0** kNm — Design resistance $M_{c,y,Rd}$ = **15.0** kNm

PASS - The bending design resistance about the y axis exceeds the design moment

Bending about z axis (cl. 6.2.5)

Design bending moment $M_{z,Ed}$ = **1.0** kNm — Design resistance $M_{c,z,Rd}$ = **15.0** kNm

PASS - The bending design resistance about the z axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about y axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed}$ = **5.0** kNm — Modified design resistance $M_{N,y,Rd}$ = **15.0** kNm

PASS - Bending resistance about y axis in presence of axial load exceeds design moment

Bending about z axis (cl. 6.2.9.1)

Design bending moment $M_{z,Ed}$ = **1.0** kNm — Modified design resistance $M_{N,z,Rd}$ = **15.0** kNm

PASS - Bending resistance about z axis in presence of axial load exceeds design moment

Biaxial bending

Section utilisation at end 1 UR_{CS_1} = **0.172** — Section utilisation at end 2 UR_{CS_2} = **0.172**

PASS - The cross-section resistance is adequate

<u>Buckling resistance (cl. 6.3)</u>

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = **116.5** kN — Flexural buck resist about z $N_{b,z,Rd}$ = **116.5** kN

Min. buckling resistance $N_{b,Rd}$ = **116.5** kN

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Design bending moment $M_{y,Ed}$ = **5.0** kNm

Lat. torsional buck length fact K_{LT} = **1.00** — Design buckling resistance mt $M_{b,Rd}$ = **15.0** kNm

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

Section utilisation UR_{B_1} = **0.642** — Section utilisation UR_{B_2} = **0.517**

PASS - The buckling resistance is adequate

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 3.0.08

Support conditions

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

Analysis results

Maximum moment	M_{max} = **273.7** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **261.4** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **273.7** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum shear	V_{max} = **118.7** kN	V_{min} = **-149.5** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **118.7** kN	$V_{s1\_seg1\_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **44.6** kN	$V_{s1\_seg2\_min}$ = **-149.5** kN
Deflection segment 3	δ_{max} = **19.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **118.7** kN	R_{A_min} = **118.7** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **63.8** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **21.8** kN	
Maximum reaction at support B	R_{B_max} = **149.5** kN	R_{B_min} = **149.5** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **82.7** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **25.3** kN	

Section details

Section type	**UC 254x254x107 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force	V_{Ed} = **150** kN	Design shear resistance	$V_{c,Rd}$ = **583** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	M_{Ed} = **273.7** kNm	Des.bending resist.moment	$M_{c,Rd}$ = **393.4** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio	$\overline{\lambda}_{LT}$ = **0.652**	Limiting slenderness ratio	$\overline{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **351.3** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

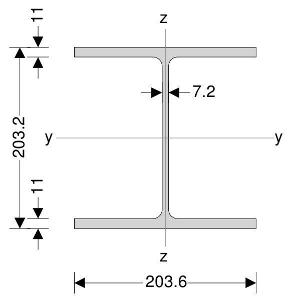
Limiting deflection	δ_{lim} = **25.6** mm	Maximum deflection	δ = **19.674** mm

PASS - Maximum deflection does not exceed deflection limit

STEEL COLUMN DESIGN (EN 1993-1-1)

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

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section **UC 203x203x46**

System length y axis buckling L_y = **3000** mm   System length z axis buckling L_z = **3000** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load N_{Ed} = **150** kN (Compression)

Moment about y axis at end 1 $M_{y,Ed1}$ = **24.0** kNm   Moment about y axis at end 2 $M_{y,Ed2}$ = **24.0** kNm

Single curvature bending about y axis

Moment about z axis at end 1 $M_{z,Ed1}$ = **1.0** kNm   Moment about z axis at end 2 $M_{z,Ed2}$ = **1.0** kNm

Single curvature bending about z axis

Shear force parallel to z axis $V_{z,Ed}$ = **1** kN   Shear force parallel to y axis $V_{y,Ed}$ = **1** kN

Material details

Steel grade **S275**

Yield strength f_y = **275** N/mm^2 Ultimate strength f_u = **410** N/mm^2

Modulus of elasticity E = **210** kN/mm^2 Shear modulus G = **80.8** kN/mm^2

Buckling length for flexural buckling about y axis

End restraint factor K_y = **1.000**   Buckling length L_{cr_y} = **3000** mm

Buckling length for flexural buckling about z axis

End restraint factor K_z = **1.000**   Buckling length L_{cr_z} = **3000** mm

Section classification (Table 5.2)

Web classification **1**   Flange classification **1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force $V_{z,Ed}$ = **1.0** kN   Plastic shear resistance $V_{pl,z,Rd}$ = **269.5** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

$V_{z,Ed}$ <= 0.5×$V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = **1.0** kN — Plastic shear resistance $V_{pl,y,Rd}$ = **663.0** kN

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed}$ <= 0.5×$V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = **150** kN — Design resistance $N_{c,Rd}$ = **1615** kN

PASS - The compression design resistance exceeds the design force

Bending about y axis (cl. 6.2.5)

Design bending moment $M_{y,Ed}$ = **24.0** kNm — Design resistance $M_{c,y,Rd}$ = **136.8** kNm

PASS - The bending design resistance about the y axis exceeds the design moment

Bending about z axis (cl. 6.2.5)

Design bending moment $M_{z,Ed}$ = **1.0** kNm — Design resistance $M_{c,z,Rd}$ = **63.5** kNm

PASS - The bending design resistance about the z axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about y axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed}$ = **24.0** kNm — Modified design resistance $M_{N,y,Rd}$ = **136.8** kNm

PASS - Bending resistance about y axis in presence of axial load exceeds design moment

Bending about z axis (cl. 6.2.9.1)

Design bending moment $M_{z,Ed}$ = **1.0** kNm — Modified design resistance $M_{N,z,Rd}$ = **63.5** kNm

PASS - Bending resistance about z axis in presence of axial load exceeds design moment

Biaxial bending

Section utilisation at end 1 UR_{CS_1} = **0.047** — Section utilisation at end 2 UR_{CS_2} = **0.047**

PASS - The cross-section resistance is adequate

<u>Buckling resistance (cl. 6.3)</u>

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = **1500.9** kN — Flexural buck resist about z $N_{b,z,Rd}$ = **1197.3** kN

Torsional buck. length factor K_T = **1.00** — Torsional/tor flex buck resist $N_{b,T,Rd}$ = **1292.0** kN

Min. buckling resistance $N_{b,Rd}$ = **1197.3** kN

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Design bending moment $M_{y,Ed}$ = **24.0** kNm

Lat. torsional buck length fact K_{LT} = **1.00** — Design buckling resistance mt $M_{b,Rd}$ = **130.7** kNm

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

Section utilisation UR_{B_1} = **0.297** — Section utilisation UR_{B_2} = **0.324**

PASS - The buckling resistance is adequate

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 1.5.08

Analysis results

Design moment M = **1.478** kNm; Design shear F = **3.940** kN

Total load on member W_{tot} = **7.880** kN

Reactions at support A R_{A_max} = **3.940** kN; R_{A_min} = **3.940** kN

Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **1.335** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **1.425** kN

Reactions at support B R_{B_max} = **3.940** kN; R_{B_min} = **3.940** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **1.335** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **1.425** kN

Timber section details

Breadth of section b = **50** mm; Depth of section h = **200** mm

Number of sections N = **1**; Breadth of member b_b = **50** mm

Timber strength class **C16**

Member details

Service class of timber **1**; Load duration **Permanent**

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.788** N/mm²; Design compressive strength $f_{c.90.d}$ = **1.015** N/mm²

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **4.433** N/mm²; Design bending strength $f_{m.d}$ = **7.385** N/mm²

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.882** N/mm²; Permissible shear stress $f_{v.d}$ = **1.477** N/mm²

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **6.000** mm; Total final deflection δ_{fin} = **1.602** mm

PASS - Total final deflection is less than the deflection limit

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A1:2008 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

TEDDS calculation version 1.5.08

Analysis results

Design moment M = **3.508** kNm; Design shear F = **7.407** kN

Total load on member W_{tot} = **10.455** kN

Reactions at support A R_{A_max} = **7.407** kN; R_{A_min} = **7.407** kN

Unfactored permanent load reaction at support A $R_{A\_Permanent}$ = **4.135** kN

Unfactored variable load reaction at support A $R_{A\_Variable}$ = **1.217** kN

Reactions at support B R_{B_max} = **3.049** kN; R_{B_min} = **3.049** kN

Unfactored permanent load reaction at support B $R_{B\_Permanent}$ = **1.188** kN

Unfactored variable load reaction at support B $R_{B\_Variable}$ = **0.963** kN

Timber section details

Breadth of section b = **50** mm; Depth of section h = **200** mm

Number of sections N = **2**; Breadth of member b_b = **100** mm

Timber strength class **C16**

Member details

Service class of timber **1**; Load duration **Permanent**

Length of bearing L_b = **100** mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c.90.d}$ = **0.741** N/mm^2; Design compressive strength $f_{c.90.d}$ = **1.015** N/mm^2

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m.d}$ = **5.263** N/mm^2; Design bending strength $f_{m.d}$ = **7.385** N/mm^2

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress τ_d = **0.829** N/mm^2; Permissible shear stress $f_{v.d}$ = **1.477** N/mm^2

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit δ_{lim} = **12.000** mm; Total final deflection δ_{fin} = **6.634** mm

PASS - Total final deflection is less than the 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 3.0.08

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = **43.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **28.2** kN	V_{min} = **-54.7** kN
Deflection	δ_{max} = **11.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **28.2** kN	R_{A_min} = **28.2** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **20.9** kN	
Maximum reaction at support B	R_{B_max} = **54.7** kN	R_{B_min} = **54.7** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **40.5** kN	

Section details

Section type **UC 152x152x30 (BS4-1)** Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = **55** kN Design shear resistance $V_{c,Rd}$ = **183.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **43.8** kNm Des.bending resist.moment $M_{c,Rd}$ = **68.1** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **1.073** Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **44.6** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection δ_{lim} = **14.4** mm Maximum deflection δ = **11.422** 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 3.0.08

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = **71.8** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **71.8** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **71.8** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum shear	V_{max} = **82** kN	V_{min} = **-82** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **82** kN	$V_{s1\_seg1\_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **0** kN	$V_{s1\_seg2\_min}$ = **-82** kN
Deflection segment 3	δ_{max} = **2.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **82** kN	R_{A_min} = **82** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **38.4** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **20.1** kN	
Maximum reaction at support B	R_{B_max} = **82** kN	R_{B_min} = **82** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **38.4** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **20.1** kN	

Section details

Section type **UC 203x203x46 (BS4-1)** — Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = **82** kN — Design shear resistance $V_{c,Rd}$ = **269.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **71.8** kNm — Des.bending resist.moment $M_{c,Rd}$ = **136.8** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **0.521** — Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **130.1** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection δ_{lim} = **9.7** mm — Maximum deflection δ = **2.343** 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 3.0.08

Support conditions

Support A — Vertically restrained; Rotationally free

Support B — Vertically restrained; Rotationally free

Analysis results

Maximum moment	M_{max} = **113.9** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **104.3** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **113.9** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum shear	V_{max} = **41.5** kN	V_{min} = **-48.8** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **41.5** kN	$V_{s1\_seg1\_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **22.7** kN	$V_{s1\_seg2\_min}$ = **-48.8** kN
Deflection segment 3	δ_{max} = **0.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **41.5** kN	R_{A_min} = **41.5** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **29.6** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **1** kN	
Maximum reaction at support B	R_{B_max} = **48.8** kN	R_{B_min} = **48.8** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **35** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **1** kN	

Section details

Section type **UC 203x203x46 (BS4-1)** Steel grade **S275**

Section classification **Class 1**

Check shear - Section 6.2.6

Design shear force V_{Ed} = **49** kN Design shear resistance $V_{c,Rd}$ = **269.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment M_{Ed} = **113.9** kNm Des.bending resist.moment $M_{c,Rd}$ = **136.8** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio $\bar{\lambda}_{LT}$ = **0.462** Limiting slenderness ratio $\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **136.8** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

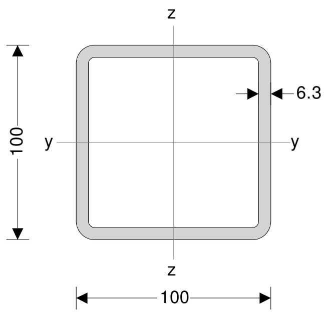
Limiting deflection δ_{lim} = **18.1** mm Maximum deflection δ = **0.727** mm

PASS - Maximum deflection does not exceed deflection limit

STEEL COLUMN DESIGN (EN 1993-1-1)

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

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section **SHS 100x100x6.3**

System length y axis buckling L_y = **3000** mm System length z axis buckling L_z = **3000** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load N_{Ed} = **50** kN (Compression)

Moment about y axis at end 1 $M_{y,Ed1}$ = **10.0** kNm Moment about y axis at end 2 $M_{y,Ed2}$ = **10.0** kNm

Single curvature bending about y axis

Moment about z axis at end 1 $M_{z,Ed1}$ = **1.0** kNm Moment about z axis at end 2 $M_{z,Ed2}$ = **1.0** kNm

Single curvature bending about z axis

Shear force parallel to z axis $V_{z,Ed}$ = **1** kN Shear force parallel to y axis $V_{y,Ed}$ = **1** kN

Material details

Steel grade **S275**

Yield strength f_y = **275** N/mm² Ultimate strength f_u = **410** N/mm²

Modulus of elasticity E = **210** kN/mm² Shear modulus G = **80.8** kN/mm²

Buckling length for flexural buckling about y axis

End restraint factor K_y = **1.000** Buckling length L_{cr_y} = **3000** mm

Buckling length for flexural buckling about z axis

End restraint factor K_z = **1.000** Buckling length L_{cr_z} = **3000** mm

Section classification (Table 5.2)

Web classification **1** Flange classification **1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force $V_{z,Ed}$ = **1.0** kN Plastic shear resistance $V_{pl,z,Rd}$ = **184.1** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

$V_{z,Ed}$ <= 0.5×$V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = **1.0** kN Plastic shear resistance $V_{pl,y,Rd}$ = **184.1** kN

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed}$ <= 0.5×$V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = **50** kN Design resistance $N_{c,Rd}$ = **638** kN

PASS - The compression design resistance exceeds the design force

Bending about y axis (cl. 6.2.5)

Design bending moment $M_{y,Ed}$ = **10.0** kNm Design resistance $M_{c,y,Rd}$ = **22.2** kNm

PASS - The bending design resistance about the y axis exceeds the design moment

Bending about z axis (cl. 6.2.5)

Design bending moment $M_{z,Ed}$ = **1.0** kNm Design resistance $M_{c,z,Rd}$ = **22.2** kNm

PASS - The bending design resistance about the z axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about y axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed}$ = **10.0** kNm Modified design resistance $M_{N,y,Rd}$ = **22.2** kNm

PASS - Bending resistance about y axis in presence of axial load exceeds design moment

Bending about z axis (cl. 6.2.9.1)

Design bending moment $M_{z,Ed}$ = **1.0** kNm Modified design resistance $M_{N,z,Rd}$ = **22.2** kNm

PASS - Bending resistance about z axis in presence of axial load exceeds design moment

Biaxial bending

Section utilisation at end 1 UR_{CS_1} = **0.269** Section utilisation at end 2 UR_{CS_2} = **0.269**

PASS - The cross-section resistance is adequate

Buckling resistance (cl. 6.3)

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = **464.5** kN Flexural buck resist about z $N_{b,z,Rd}$ = **464.5** kN

Min. buckling resistance $N_{b,Rd}$ = **464.5** kN

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Design bending moment $M_{y,Ed}$ = **10.0** kNm

Lat. torsional buck length fact K_{LT} = **1.00** Design buckling resistance mt $M_{b,Rd}$ = **22.2** kNm

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

Section utilisation UR_{B_1} = **0.621** Section utilisation UR_{B_2} = **0.446**

PASS - The buckling resistance is adequate

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 3.0.10

Support conditions

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

Analysis results

Maximum moment	M_{max} = **270.9** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **258.8** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **270.9** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum shear	V_{max} = **117.9** kN	V_{min} = **-99** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **117.9** kN	$V_{s1\_seg1\_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **43.8** kN	$V_{s1\_seg2\_min}$ = **-99** kN
Deflection segment 3	δ_{max} = **19.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **117.9** kN	R_{A_min} = **117.9** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **63.2** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **21.8** kN	
Maximum reaction at support B	R_{B_max} = **99** kN	R_{B_min} = **99** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **45.3** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **25.3** kN	

Section details

Section type	**UC 254x254x107 (BS4-1)**	Steel grade	**S275**
Section classification	**Class 1**		

Check shear - Section 6.2.6

Design shear force	V_{Ed} = **118** kN	Design shear resistance	$V_{c,Rd}$ = **583** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment	M_{Ed} = **270.9** kNm	Des.bending resist.moment	$M_{c,Rd}$ = **393.4** kNm

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio	$\bar{\lambda}_{LT}$ = **0.652**	Limiting slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.400**

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

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment $M_{b,Rd}$ = **351.3** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

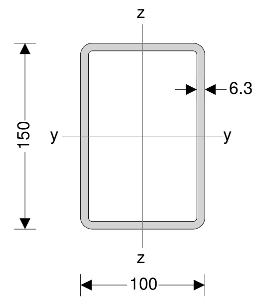
Limiting deflection	δ_{lim} = **25.6** mm	Maximum deflection	δ = **19.41** mm

PASS - Maximum deflection does not exceed deflection limit

STEEL COLUMN DESIGN (EN 1993-1-1)

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

TEDDS calculation version 1.0.09



Column and loading details

Column details

Column section **RHS 150x100x6.3**

System length y axis buckling L_y = **3000** mm System length z axis buckling L_z = **3000** mm

Sway

The column is not part of a sway frame in the direction of the z axis

The column is not part of a sway frame in the direction of the y axis

Column loading

Axial load N_{Ed} = **120** kN (Compression)

Moment about y axis at end 1 $M_{y,Ed1}$ = **24.0** kNm Moment about y axis at end 2 $M_{y,Ed2}$ = **24.0** kNm

Single curvature bending about y axis

Moment about z axis at end 1 $M_{z,Ed1}$ = **1.0** kNm Moment about z axis at end 2 $M_{z,Ed2}$ = **1.0** kNm

Single curvature bending about z axis

Shear force parallel to z axis $V_{z,Ed}$ = **1** kN Shear force parallel to y axis $V_{y,Ed}$ = **1** kN

Material details

Steel grade **S275**

Yield strength f_y = **275** N/mm² Ultimate strength f_u = **410** N/mm²

Modulus of elasticity E = **210** kN/mm² Shear modulus G = **80.8** kN/mm²

Buckling length for flexural buckling about y axis

End restraint factor K_y = **1.000** Buckling length L_{cr_y} = **3000** mm

Buckling length for flexural buckling about z axis

End restraint factor K_z = **1.000** Buckling length L_{cr_z} = **3000** mm

Section classification (Table 5.2)

Web classification **1** Flange classification **1**

The section is class 1

Resistance of cross section (cl. 6.2)

Shear parallel to z axis (cl. 6.2.6)

Design shear force $V_{z,Ed}$ = **1.0** kN Plastic shear resistance $V_{pl,z,Rd}$ = **280.9** kN

PASS - Shear resistance parallel to z axis exceeds the design shear force

$V_{z,Ed}$ <= 0.5×$V_{pl,z,Rd}$ - No reduction in f_y required for bending/axial force

Shear parallel to y axis (cl. 6.2.6)

Design shear force $V_{y,Ed}$ = **1.0** kN Plastic shear resistance $V_{pl,y,Rd}$ = **187.3** kN

PASS - Shear resistance parallel to y axis exceeds the design shear force

$V_{y,Ed}$ <= 0.5×$V_{pl,y,Rd}$ - No reduction in f_y required for bending/axial force

Compression (cl. 6.2.4)

Design force N_{Ed} = **120** kN Design resistance $N_{c,Rd}$ = **811** kN

PASS - The compression design resistance exceeds the design force

Bending about y axis (cl. 6.2.5)

Design bending moment $M_{y,Ed}$ = **24.0** kNm Design resistance $M_{c,y,Rd}$ = **40.3** kNm

PASS - The bending design resistance about the y axis exceeds the design moment

Bending about z axis (cl. 6.2.5)

Design bending moment $M_{z,Ed}$ = **1.0** kNm Design resistance $M_{c,z,Rd}$ = **30.4** kNm

PASS - The bending design resistance about the z axis exceeds the design moment

Combined bending and axial force (cl. 6.2.9)

Bending about y axis (cl. 6.2.9.1)

Design bending moment $M_{y,Ed}$ = **24.0** kNm Modified design resistance $M_{N,y,Rd}$ = **40.3** kNm

PASS - Bending resistance about y axis in presence of axial load exceeds design moment

Bending about z axis (cl. 6.2.9.1)

Design bending moment $M_{z,Ed}$ = **1.0** kNm Modified design resistance $M_{N,z,Rd}$ = **30.4** kNm

PASS - Bending resistance about z axis in presence of axial load exceeds design moment

Biaxial bending

Section utilisation at end 1 UR_{CS_1} = **0.416** Section utilisation at end 2 UR_{CS_2} = **0.416**

PASS - The cross-section resistance is adequate

Buckling resistance (cl. 6.3)

Axial buckling resistance

Flexural buck resist about y $N_{b,y,Rd}$ = **713.4** kN Flexural buck resist about z $N_{b,z,Rd}$ = **615.1** kN

Min. buckling resistance $N_{b,Rd}$ = **615.1** kN

PASS - The axial load buckling resistance exceeds the design axial load

Buckling resistance moment (cl.6.3.2.1)

Design bending moment $M_{y,Ed}$ = **24.0** kNm

Lat. torsional buck length fact K_{LT} = **1.00** Design buckling resistance mt $M_{b,Rd}$ = **40.3** kNm

PASS - The design buckling resistance moment exceeds the maximum design moment

Combined bending and axial compression (cl. 6.3.3)

Section utilisation UR_{B_1} = **0.828** Section utilisation UR_{B_2} = **0.814**

PASS - The buckling resistance is adequate