PSC Paul Snape Consulting The Granary, Woodfold Farm Crombleholme Fold, Goosnargh Preston PR3 2ES pa.snape@outlook.com	Project Picket Post House, Claughton on Brock, PR3 0PL				Job Ref. PSC-628	
	Section External Balcony Design				Sheet no./rev. 1	
	Calc. by PAS	Date 28/03/2022	Chk'd by PAS	Date 28/03/22	App'd by PAS	Date 28/03/22

INTRODUCTION

Contractor requested a design for a 3m wide and 2.8m deep external balcony at picket post house. Design set out below for required joists, timber or steel beam to support joists (the joists will be supported on the external wall at one end) and timber or steel columns to support the beam. A baseplate design is provided for the steel column. If timber columns are used then a shoe to houise the timber will need to be welded onto the baseplate. The joists will be supported on the external wall.

Loading is as per domestic floor with dead load of 0.75 kN/m^2 and imposed lopad of 1.5 kN/m^2.

TIMBER JOISTS TO BALCONY

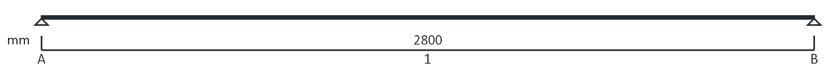
Joists span 2.8m

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

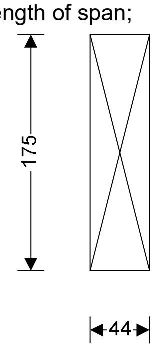
Joist details

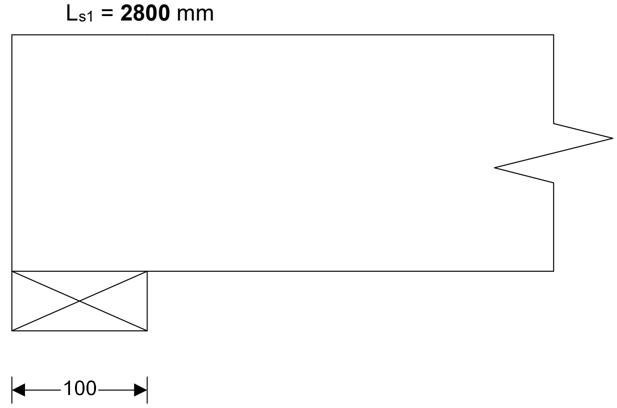
Joist breadth;	b = **44** mm
Joist depth;	h = **175** mm
Joist spacing;	s = **400** mm
Timber strength class;	**C16**
Service class of timber;	**1**



Span details

Number of spans;	N_{span} = **1**
Length of bearing;	L_b = **100** mm
Effective length of span;	L_{s1} = **2800** mm





Section properties

Second moment of area;	$I = b \times h^3 / 12 =$ **19651042** mm^4
Section modulus;	$Z = b \times h^2 / 6 =$ **224583** mm^3

Loading details

Joist self weight; $F_{swt} = b \times h \times \rho_{char} \times g_{acc} = \mathbf{0.02}$ kN/m

Dead load; $F_{d\_udl} = \mathbf{1.10}$ kN/m^2

Imposed UDL(Long term); $F_{i\_udl} = \mathbf{1.50}$ kN/m^2

Imposed point load (Medium term); $F_{i\_pt} = \mathbf{1.40}$ kN

Modification factors

Service class for bending parallel to grain $K_{2m} = \mathbf{1.00}$

Service class for compression $K_{2c} = \mathbf{1.00}$

Service class for shear parallel to grain $K_{2s} = \mathbf{1.00}$

Service class for modulus of elasticity $K_{2e} = \mathbf{1.00}$

Section depth factor; $K_7 = \mathbf{1.06}$

Load sharing factor; $K_8 = \mathbf{1.10}$

Consider long term loads

Load duration factor; $K_3 = \mathbf{1.00}$

Maximum bending moment; $M = \mathbf{1.042}$ kNm

Maximum shear force; $V = \mathbf{1.489}$ kN

Maximum support reaction; $R = \mathbf{1.489}$ kN

Maximum deflection; $\delta = \mathbf{5.217}$ mm

Check bending stress

Bending stress; $\sigma_m = \mathbf{5.300}$ N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = \mathbf{6.186}$ N/mm^2

Applied bending stress; $\sigma_{m\_max} = M / Z = \mathbf{4.640}$ N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; $\tau = \mathbf{0.670}$ N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = \mathbf{0.737}$ N/mm^2

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h) = \mathbf{0.290}$ N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane); $\sigma_{cp1} = \mathbf{2.200}$ N/mm^2

Permissible bearing stress; $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = \mathbf{2.420}$ N/mm^2

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b) = \mathbf{0.338}$ N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = \mathbf{8.400}$ mm

Bending deflection (based on E_{mean}); $\delta_{bending} = \mathbf{4.922}$ mm

Shear deflection; $\delta_{shear} = \mathbf{0.295}$ mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear} = \mathbf{5.217}$ mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor; $K_3 = \mathbf{1.25}$

Maximum bending moment; $M = \mathbf{1.434}$ kNm

Maximum shear force; $V = \mathbf{2.049}$ kN

Maximum support reaction; $R = \mathbf{2.049}$ kN

Maximum deflection; $\delta = \mathbf{6.254}$ mm

Check bending stress

Bending stress; σ_m = **5.300** N/mm^2

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8$ = **7.733** N/mm^2

Applied bending stress; $\sigma_{m\_max} = M / Z$ = **6.386** N/mm^2

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress; τ = **0.670** N/mm^2

Permissible shear stress; $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8$ = **0.921** N/mm^2

Applied shear stress; $\tau_{max} = 3 \times V / (2 \times b \times h)$ = **0.399** N/mm^2

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane); σ_{cp1} = **2.200** N/mm^2

Permissible bearing stress; $\sigma_{c\_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8$ = **3.025** N/mm^2

Applied bearing stress; $\sigma_{c\_max} = R / (b \times L_b)$ = **0.466** N/mm^2

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm})$ = **8.400** mm

Bending deflection (based on E_{mean}); $\delta_{bending}$ = **5.847** mm

Shear deflection; δ_{shear} = **0.406** mm

Total deflection; $\delta = \delta_{bending} + \delta_{shear}$ = **6.254** mm

PASS - Actual deflection within permissible limits

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TIMBER BEAM TO BALCONY

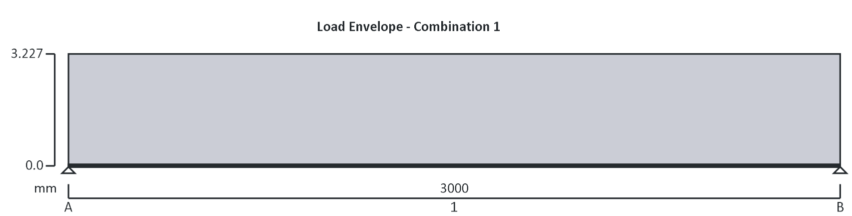
Beam spans 3m and carries 1.4m of floor load.

Dead load = 1.4x0.75 = 1.05 kN/m

Imposed load = 1.4x1.5 = 2.1 kN/m

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02



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	Section External Balcony Design				Sheet no./rev. 4	
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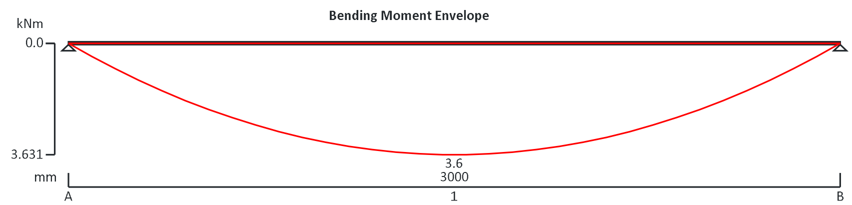
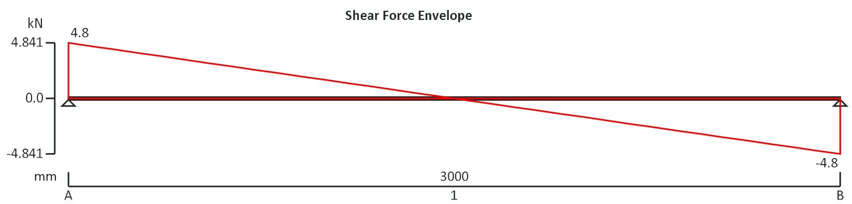




Applied loading

Beam loads

Dead self weight of beam × 1

Dead full UDL 1.050 kN/m

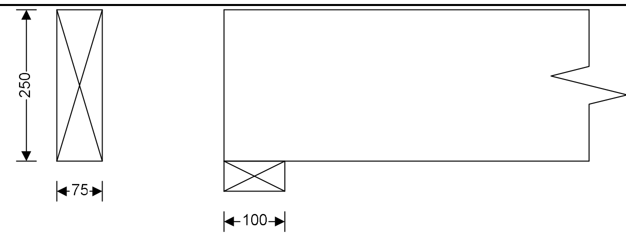
Imposed full UDL 2.100 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.00
		Imposed × 1.00
	Span 1	Dead × 1.00
		Imposed × 1.00
	Support B	Dead × 1.00
		Imposed × 1.00

Analysis results

Maximum moment;	M_{max} = **3.631** kNm;	M_{min} = **0.000** kNm
Design moment;	M = max(abs(M_{max}),abs(M_{min})) = **3.631** kNm	
Maximum shear;	F_{max} = **4.841** kN;	F_{min} = **-4.841** kN
Design shear;	F = max(abs(F_{max}),abs(F_{min})) = **4.841** kN	
Total load on beam;	W_{tot} = **9.682** kN	
Reactions at support A;	R_{A_max} = **4.841** kN;	R_{A_min} = **4.841** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **1.691** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **3.150** kN	
Reactions at support B;	R_{B_max} = **4.841** kN;	R_{B_min} = **4.841** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **1.691** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **3.150** kN	



Timber section details

Breadth of sections; $b = \mathbf{75}$ mm

Depth of sections; $h = \mathbf{250}$ mm

Number of sections in member; $N = \mathbf{1}$

Overall breadth of member; $b_b = N \times b = \mathbf{75}$ mm

Timber strength class; **C24**

Member details

Service class of timber; **1**

Load duration; **Long term**

Length of span; $L_{s1} = \mathbf{3000}$ mm

Length of bearing; $L_b = \mathbf{100}$ mm

Section properties

Cross sectional area of member; $A = N \times b \times h = \mathbf{18750}$ mm^2

Section modulus; $Z_x = N \times b \times h^2 / 6 = \mathbf{781250}$ mm^3

$Z_y = h \times (N \times b)^2 / 6 = \mathbf{234375}$ mm^3

Second moment of area; $I_x = N \times b \times h^3 / 12 = \mathbf{97656250}$ mm^4

$I_y = h \times (N \times b)^3 / 12 = \mathbf{8789062}$ mm^4

Radius of gyration; $i_x = \sqrt{(I_x / A)} = \mathbf{72.2}$ mm

$i_y = \sqrt{(I_y / A)} = \mathbf{21.7}$ mm

Modification factors

Duration of loading - Table 17; $K_3 = \mathbf{1.00}$

Bearing stress - Table 18; $K_4 = \mathbf{1.00}$

Total depth of member - cl.2.10.6; $K_7 = (300 \text{ mm} / h)^{0.11} = \mathbf{1.02}$

Load sharing - cl.2.9; $K_8 = \mathbf{1.00}$

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by purlins or tie rods at centres not more than 30 times the breadth of the member

Permissible depth-to-breadth ratio - Table 19; **4.00**

Actual depth-to-breadth ratio; $h / (N \times b) = \mathbf{3.33}$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane); $\sigma_{c\_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = \mathbf{2.400}$ N/mm^2

Applied bearing stress; $\sigma_{c\_a} = R_{A\_max} / (N \times b \times L_b) = \mathbf{0.645}$ N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm} = \mathbf{0.269}$

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{7.652}$ N/mm^2

Applied bending stress; $\sigma_{m\_a} = M / Z_x =$ **4.647** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.607**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.710** N/mm^2

Applied shear stress; $\tau_a = 3 \times F / (2 \times A) =$ **0.387** N/mm^2

$\tau_a / \tau_{adm} =$ **0.545**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection; $E = E_{min} =$ **7200** N/mm^2

Permissible deflection; $\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) =$ **9.000** mm

Bending deflection; $\delta_{b\_s1} =$ **4.841** mm

Shear deflection; $\delta_{v\_s1} =$ **0.516** mm

Total deflection; $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} =$ **5.357** mm

$\delta_a / \delta_{adm} =$ **0.595**

PASS - Total deflection is less than permissible deflection

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STEEL BEAM TO BALCONY

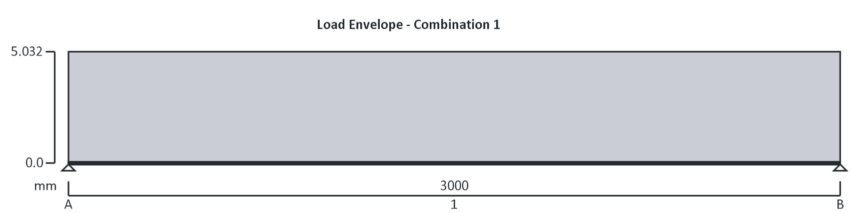
Loading and span as above for timber.

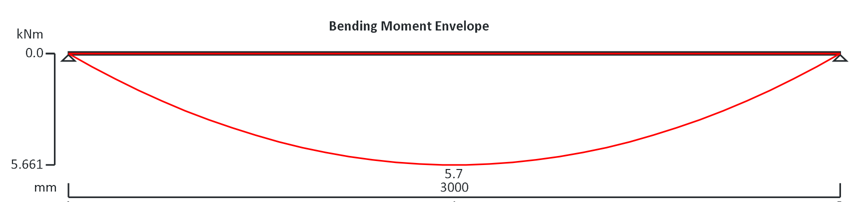
Design as unrestrained.

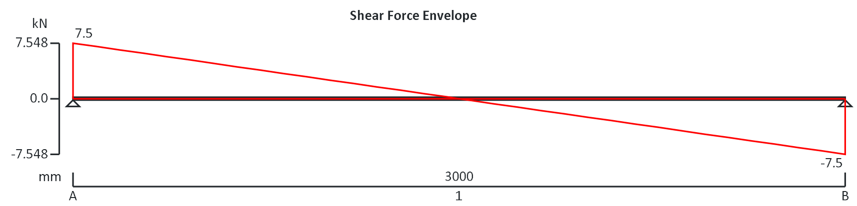
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 1.05 kN/m
	Imposed full UDL 2.1 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

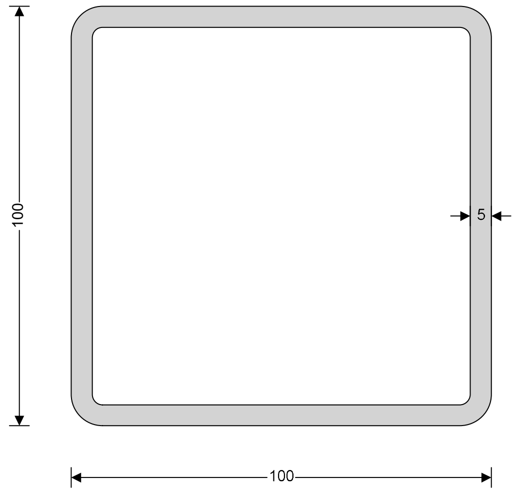
Maximum moment;	M_{max} = **5.7** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **7.5** kN;	V_{min} = **-7.5** kN
Deflection;	δ_{max} = **3.9** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **7.5** kN;	R_{A_min} = **7.5** kN
Unfactored dead load reaction at support A;	R_{A_Dead} = **1.8** kN	
Unfactored imposed load reaction at support A;	$R_{A\_Imposed}$ = **3.2** kN	
Maximum reaction at support B;	R_{B_max} = **7.5** kN;	R_{B_min} = **7.5** kN
Unfactored dead load reaction at support B;	R_{B_Dead} = **1.8** kN	
Unfactored imposed load reaction at support B;	$R_{B\_Imposed}$ = **3.2** kN	

Section details

Section type;	**SHS 100x100x5.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))**
Steel grade;	**S275**

From table 9: Design strength p_y

Thickness of element;	t = **5.0** mm
Design strength;	p_y = **275** N/mm^2
Modulus of elasticity;	E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; $K_{LT.A}$ = **1.00**;

$K_{LT.B}$ = **1.00**;

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.00**

Web - major axis - Table 12

Depth of section; $d = D - 3 \times t$ = **85** mm

$d / t = 17.0 \times \varepsilon <= 64 \times \varepsilon$; Class 1 plastic

Flange - major axis - Table 12

Width of section; $b = B - 3 \times t$ = **85** mm

$b / t = 17.0 \times \varepsilon <= \min(28 \times \varepsilon, 80 \times \varepsilon - d / t)$; Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **7.5** kN

$(D - 3 \times t) / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = A \times D / (D + B)$ = **937** mm^2

Design shear resistance; $P_v = 0.6 \times p_y \times A_v$ = **154.5** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **5.7** kNm

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S, 1.2 \times p_y \times Z)$ = **18.2** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1}$ = **3000** mm

Slenderness ratio; $\lambda = L_E / r_{yy}$ = **77.673**

Equivalent slenderness - Annex B.2.6.1

Torsion constant; J = **4394108** mm^4

$\gamma_b = (1 - I_{yy} / I_{xx}) \times (1 - J / (2.6 \times I_{xx}))$ = **0.000**

$\phi_b = [S_{xx}^2 \times \gamma_b / (A \times J)]^{0.5}$ = **0.000**

Ratio - cl.4.3.6.9; β_W = **1.000**

Equivalent slenderness; $\lambda_{LT} = 2.25 \times \sqrt{[\phi_b \times \lambda \times \beta_W]}$ = **0.000**

Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **34.310**

$\lambda_{LT} < \lambda_{L0}$ - No allowance need be made for lateral-torsional buckling

Buckling resistance moment - Section 4.3.6.4

Bending strength; $p_b = p_y$ = **275** N/mm^2

Buckling resistance moment; $M_b = p_b \times S$ = **18.2** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360$ = **8.333** mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **3.866** mm

PASS - Maximum deflection does not exceed deflection limit

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TIMBER COLUMN

From calculations above axial load is 7.5 kN compression. Allow for 2.6m length. Nominal moment and shear applied.

TIMBER MEMBER DESIGN TO BS5268-2:2002

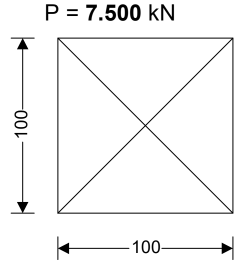
TEDDS calculation version 1.7.02

Analysis results

Design moment in major axis; M_x = **1.000** kNm

Design shear; F = **1.000** kN

Design axial compression; P = **7.500** kN



Timber section details

Breadth of sections; b = **100** mm

Depth of sections; h = **100** mm

Number of sections in member; N = **1**

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

Timber strength class; **C24**

Member details

Service class of timber; **1**

Load duration; **Long term**

Effective length - cl.2.11.3

Unbraced length in x-axis; L_x = **2600** mm
Effective length factor in x-axis - Table 21; K_x = **1**
Effective length in x-axis; $L_{ex} = L_x \times K_x$ = **2600** mm
Unbraced length in y-axis; L_y = **2600** mm
Effective length factor in y-axis - Table 21; K_y = **1**
Effective length in y-axis; $L_{ey} = L_y \times K_y$ = **2600** mm

Section properties

Cross sectional area of member; $A = N \times b \times h$ = **10000** mm^2
Section modulus; $Z_x = N \times b \times h^2 / 6$ = **166667** mm^3
$Z_y = h \times (N \times b)^2 / 6$ = **166667** mm^3
Second moment of area; $I_x = N \times b \times h^3 / 12$ = **8333333** mm^4
$I_y = h \times (N \times b)^3 / 12$ = **8333333** mm^4
Radius of gyration; $i_x = \sqrt{(I_x / A)}$ = **28.9** mm
$i_y = \sqrt{(I_y / A)}$ = **28.9** mm

Modification factors

Duration of loading - Table 17; K_3 = **1.00**
Total depth of member - cl.2.10.6; $K_7 = (300\text{ mm} / h)^{0.11}$ = **1.13**
Load sharing - cl.2.9; K_8 = **1.00**
Members subject to axial compression - Table 22; K_{12} = **0.46**

Lateral support - cl.2.10.8

No lateral support
Permissible depth-to-breadth ratio - Table 19; **2.00**
Actual depth-to-breadth ratio; $h / (N \times b)$ = **1.00**

PASS - Lateral support is adequate

Slenderness ratio - cl.2.11.4

Permissible slenderness ratio; λ_{max} = **180**
Slenderness ratio; $\lambda = \max(L_{ex} / i_x, L_{ey} / i_y)$ = **90.067**

PASS - Slenderness ratio is less than permissible slenderness ratio

Bending parallel to grain

Permissible bending stress; $\sigma_{m\_adm} = \sigma_m \times K_3 \times K_7 \times K_8$ = **8.463** N/mm^2
Applied bending stress; $\sigma_{m\_a} = M_x / Z_x$ = **6.000** N/mm^2
$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.709**

PASS - Applied bending stress is less than permissible bending stress

Compression parallel to grain

Permissible compressive stress; $\sigma_{c\_adm} = \sigma_c \times K_3 \times K_8 \times K_{12}$ = **3.618** N/mm^2
Applied compressive stress; $\sigma_{c\_a} = P / A$ = **0.750** N/mm^2
$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.207**

PASS - Applied compressive stress is less than permissible compressive stress

Members subject to axial compression and bending - cl.2.11.6

Euler critical stress; $\sigma_e = (\pi^2 \times E_{min}) / \lambda^2$ = **8.760** N/mm^2
Euler coefficient; $K_{eu} = 1 - (1.5 \times \sigma_{c\_a} \times K_{12} / \sigma_e)$ = **0.941**
Combined axial compression and bending check; $\sigma_{m\_a} / (\sigma_{m\_adm} \times K_{eu}) + \sigma_{c\_a} / \sigma_{c\_adm}$ = **0.961**; **< 1**

PASS - Combined compressive and bending stresses are within permissible limits

Shear parallel to grain

Permissible shear stress; $\tau_{adm} = \tau \times K_3 \times K_8 =$ **0.710** N/mm^2

Applied shear stress; $\tau_a = 3 \times F / (2 \times A) =$ **0.150** N/mm^2

$\tau_a / \tau_{adm} =$ **0.211**

PASS - Applied shear stress is less than permissible shear stress

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STEEL COLUMN

Details as above for timber.

STEEL MEMBER DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

Section details

Section type; **SHS 100x100x5.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))**

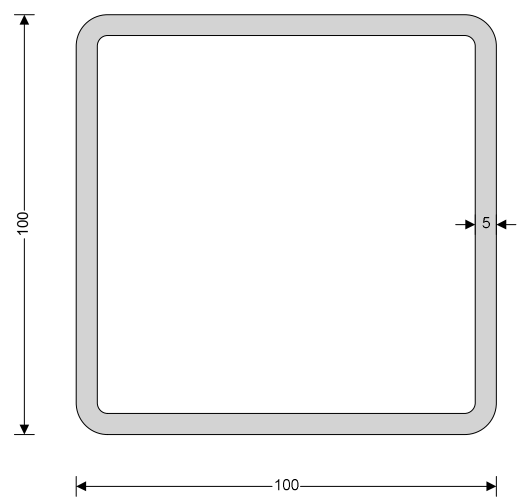
Steel grade; **S275**

From table 9: Design strength p_y

Thickness of element; t = **5.0** mm

Design strength; p_y = **275** N/mm^2

Modulus of elasticity; E = **205000** N/mm^2



Lateral restraint

Distance between major axis restraints; L_x = **2600** mm

Distance between minor axis restraints; L_y = **0** mm

Effective length factors

Effective length factor in major axis; K_x = **1.00**

Effective length factor in minor axis; K_y = **1.00**

Effective length factor for lateral-torsional buckling; K_{LT} = **1.00**;

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} =$ **1.00**

Web - major axis - Table 12

Depth of section; $d = D - 3 \times t =$ **85** mm

Stress ratios; $r1 = \min(F_c / (2 \times d \times t \times p_{yw}), 1) =$ **0.032**

$r2 = F_c / (A \times p_{yw}) =$ **0.015**

$d / t = 17.0 \times \varepsilon <= \max(64 \times \varepsilon / (1 + r1), 40 \times \varepsilon)$; Class 1 plastic

Flange - major axis - Table 12

Width of section; $b = B - 3 \times t =$ **85** mm

$b / t = 17.0 \times \varepsilon <= 40 \times \varepsilon$; Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force; $F_{y,v} =$ **1** kN

$(D - 3 \times t) / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = A \times D / (D + B) =$ **937** mm^2

Design shear resistance; $P_{y,v} = 0.6 \times p_y \times A_v =$ **154.5** kN

PASS - Design shear resistance exceeds design shear force

Shear capacity - Section 4.2.3

Design shear force; $F_{x,v} =$ **0** kN

Moment capacity - Section 4.2.5

Design bending moment; $M =$ **1** kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange; $\beta_{2f} = \min(32 \times \varepsilon, 62 \times \varepsilon - 0.5 \times d / t) =$ **32**

Limiting value for class 3 semi-compact flange; $\beta_{3f} = 40 \times \varepsilon =$ **40**

Limiting value for class 2 compact web; $\beta_{2w} = \max(80 \times \varepsilon / (1 + r1), 40 \times \varepsilon) =$ **77.513**

Limiting value for class 3 semi-compact web; $\beta_{3w} = \max(120 \times \varepsilon / (1 + 2 \times r2), 40 \times \varepsilon) =$ **116.605**

Effective plastic modulus - cl.3.5.6.3

$S_{eff} = \min(Z + (S - Z) \times \min([(\beta_{3w} / (d / t) - 1) / (\beta_{3w} / \beta_{2w} - 1)], [(\beta_{3f} / (b / t) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S) =$ **66358** mm^3

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z) =$ **18.2** kNm

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force; $F_c =$ **7.5** kN

Effective length for major (x-x) axis buckling - Section 4.7.3

Effective length for buckling; $L_{Ex} = L_x \times K_x =$ **2600** mm

Slenderness ratio - cl.4.7.2; $\lambda_x = L_{Ex} / r_{xx} =$ **67.317**

Compressive strength - Section 4.7.5

Limiting slenderness; $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5} =$ **17.155**

Strut curve - Table 23; a

Robertson constant; $\alpha_x =$ **2.0**

Perry factor; $\eta_x = \alpha_x \times (\lambda_x - \lambda_0) / 1000 =$ **0.100**

Euler stress; $p_{Ex} = \pi^2 \times E / \lambda_x^2 =$ **446.5** N/mm^2

$\phi_x = (p_y + (\eta_x + 1) \times p_{Ex}) / 2 =$ **383.1** N/mm^2

Compressive strength - Annex C.1; $p_{cx} = p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5}) =$ **228.2** N/mm^2

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4; $P_{cx} = A \times p_{cx} =$ **427.4** kN

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comb.compression & bending check - cl.4.8.3.2; $F_c / (A \times p_y) + M / M_c$ = **0.069**

PASS - Combined bending and compression check is satisfied

Member buckling resistance - Section 4.8.3.3

Max major axis moment governing M_b; $M_{LT} = M_x$ = **1.00** kNm

Equiv uniform mnt factor - major axis flex buckling; m_x = **1.000**

Buckling resistance check - cl.4.8.3.3.3; $F_c / P_{cx} + m_x \times M / M_c \times (1 + 0.5 \times F_c / P_{cx})$ = **0.073**

PASS - Member buckling resistance checks are satisfied

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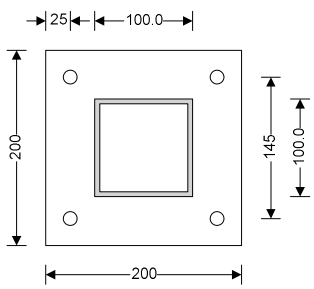
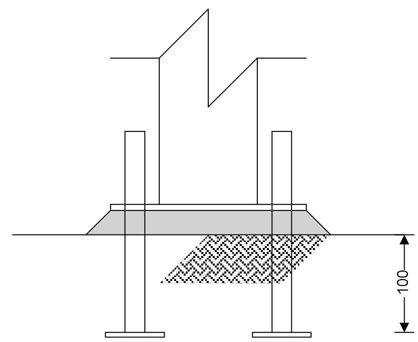
COLUMN BASE PLATE DESIGN (BS5950)

Load as per column above – 7.5 kN

COLUMN BASE PLATE DESIGN (BS5950-1:2000)

TEDDS calculation version 1.0.09;





6 mm thick base plate

Base plate reference; **Column**;

Design forces and moments

Axial force; F_c = ;**7.5**; kN (Compression)

Bending moment; M = **1.0** kNm; (about major axis)

Shear force; F_v = **1.0** kN

Column details

Column section; **SHS 100x100x5.0 (Grade S275)**

Depth; D = **100.0** mm

Breadth; B = **100.0** mm

Flange thickness; T = **5.0** mm

Web thickness; t = **5.0** mm

Design strength; p_{yc} = **275** N/mm^2

Column flange to base plate weld; **6 mm FW**;

Column web to base plate weld; **8 mm FW**;

Baseplate details

Steel grade;	**S275**
Depth;	D_p = **200** mm
Breadth;	B_p = **200** mm
Thickness;	t_p = **6** mm
Design strength;	p_{yp} = **275** N/mm^2

Holding down bolt and anchor plate details

Total number of bolts;	**4 No. M12 Grade 4.6**
Bolt spacing;	s_{bolt} = ;**145**; mm
Edge distance;	e_1 = **25** mm
Anchor plate steel grade;	**S275**
Anchor plate dimension (square);	b_{ap} = **60** mm
Anchor plate thickness;	t_{ap} = **5** mm
Design strength;	p_{yap} = **275** N/mm^2
Embeddment to top of anchor plate;	E = **100** mm
Characteristic strength of concrete;	f_{cu} = **25** N/mm^2

Concrete compression force and bolt tension force

Plate overhang beyond face of flange;	$L_1 = (D_p - D)/2)$ = ;**50.0**; mm
Effective width of plate;	$B_{pc} = \min(B_p, B + 2\times L_1)$ = ;**200.0**; mm
Distance from bolts to compression edge;	$h = D_p - e_1$ = **175** mm

Assuming a rectangular compression block of width b_{pc}, length x and intensity $0.6f_{cu}$ then:-

From static equilibrium;	$M = 0.6f_{cu}B_{pc}x(h-x/2) - F_c(h-D_p/2)$
Rearranging the quadratic equation;	$0.3f_{cu}B_{pc}x^2 - 0.6f_{cu}B_{pc}hx + F_c(h-D_p/2) + M = 0$
Factor a;	$a = 0.3 \times f_{cu} \times B_{pc}$ = **1500.0** N/mm
Factor b;	$b = -0.6 \times f_{cu} \times B_{pc} \times h$ = **-525000.0** N
Constant c;	$c = F_c \times (h-D_p/2) + M$ = **1562500.0** Nmm
Depth of compression block;	$x = [-1.0\times b - \sqrt{(b^2 - 4\times a\times c)}]/(2\times a)$ = **3.0** mm
Compression force in concrete;	$C_f = 0.6 \times f_{cu} \times B_{pc} \times x$ = **9.0** kN
Tension force in bolts;	$T_f = C_f - F_c$ = **1.5** kN

Therefore the bolts are in tension

Compression side bending

Moment in plate;	$m_c = 0.6 \times f_{cu} \times x \times (L_1 - 0.8\times s_{wf} - x/2)$ = ;**1968 Nmm/mm**;
Plate thickness required;	$t_{pc} = \sqrt{(4 \times m_c/p_{yp})}$ = **5.3** mm

Tension side bending

Lever arm;	$m = L_1 - e_1 - 0.8\times s_{wf}$ = **20.2** mm
Moment in plate;	$m_t = T_f \times m$ = **30417** Nmm
Distance from bolt cl. to face of column;	$L_f = L_1 - e_1$ = **25.0** mm
Effective plate width;	$B_{pt} = \min(B_p, s_{bolt}\times(N_{bolt}/2-1) + 2\times L_f)$ = **195.0** mm
Plate thickness required;	$t_{pt} = \sqrt{(4 \times m_t/(p_{yp}\times B_{pt}))}$ = **1.5** mm

Plate thickness

Plate thickness required;	$t_{p\_req} = \max(t_{pc}, t_{pt})$ = ;**5.3 mm**;
Plate thickness provided;	t_p = **6** mm

PASS - Plate thickness provided is adequate (0.892)

Flange weld

Tension capacity of flange;	$P_{tf} = B \times T \times p_{yc}$ = ;**137.5**; kN
Force in tension flange;	$F_{tf} = M/(D - T) - F_c \times (B\times T)/A$ = ;**8.5**; kN

Flange weld design force;	$F_f = \min(P_{tf}, \max(F_{tf}, 0\text{ kN})) = \mathbf{8.5}$ kN
Weld force per mm;	$f_{wf} = F_f/B$ = ;**0.085**; kN/mm
Transverse capacity of 6 mm fillet weld;	p_{wf} = **1.155** kN/mm; (Cl. 6.8.7.3)

PASS - Flange weld capacity is adequate (0.074)

Longitudinal capacity of web weld

Weld force per mm;	$f_{wwl} = F_v/(2\times(D-2\times t))$ = ;**0.006**; kN/mm
Longitudinal capacity of 8 mm fillet weld;	p_{wwl} = **1.232** kN/mm; (Cl. 6.8.7.3)

PASS - Longitudinal capacity of web weld is adequate (0.005)

Holding down bolts

Force per bolt;	$F_{bolt} = (2\times T_f)/ N_{bolt}$ = **0.8** kN
Tensile area per bolt;	A_{t_b} = **84.3** mm^2
Tensile strength;	p_{t_b} = **240** N/mm^2
Tension capacity (cl. 6.6);	$P_{t\_b} = 0.8 \times p_{t\_b} \times A_{t\_b}$ = ;**16.2**; kN;

PASS - Bolt capacity is adequate (0.047)

Anchor plates

Force per anchor plate;	$F_{ap} = F_{bolt}$ = **0.8** kN
Bolt hole diameter in anchor plate;	d_h = **13** mm
Anchor plate bearing area;	$A_{ap} = b_{ap}^2 - \pi\times d_h^2/4$ = **3467** mm^2
Bearing capacity;	$P_{ap} = 0.6 \times f_{cu} \times A_{ap}$ = **52.0** kN

PASS - Anchor plate bearing capacity is adequate (0.014)

Bearing pressure on anchor plate;	$f_{ap} = F_{ap} / A_{ap}$ = **0.2** N/mm^2
Width of bolt head (across flats);	d_{bh} = **19.0** mm
Maximum cantilever length;	$l_{ap} = b_{ap}/2 \times \sqrt{(2)} - d_{bh}/2$ = **32.9** mm
Bending moment in plate;	$m_{ap} = f_{ap} \times l_{ap}^2/2$ = **0.1** Nm/mm
Bending capacity;	$m_{cap} = p_{yap} \times t_{ap}^2/4$ = **1.7** Nm/mm

PASS - Anchor plate bending capacity is adequate (0.068)

Holding down bolt anchorage

Note - the following calculation to check the holding down bolt anchorage into the foundation assumes that the edges of the foundation are sufficiently far from the anchor plates to not affect the punching shear perimeter.

Tension force to be resisted;	$F_t = T_f$ = **1.5** kN
Nominal cover to top reinforcement;	c_{nom} = **30** mm
Effective depth of HD bolts;	$L_{HD} = E - c_{nom}$ = **70** mm
Shear strength of concrete;	v_c = **0.34** N/mm^2
Effective shear perimeter;	$P_{HD} = 2 \times [(b_{ap} + 2\times1.5\times L_{HD}) + (s_{bolt} \times (N_{bolt}/2 - 1) + b_{ap} + 2\times1.5\times L_{HD})]$
	P_{HD} = **1370** mm
Pull-out capacity of tension bolts;	$P_t = v_c \times P_{HD} \times L_{HD}$ = **32.6** kN

PASS - Holding down bolt anchorage is adequate (0.046)

Shear transfer to concrete

Assumed coefficient of friction;	$\mu = 0.30$
Available shear resistance;	$P_v = C_f \times \mu$ = ;**3**; kN

PASS - Frictional shear capacity is adequate (0.370)

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