

 Hughes Consulting Ltd Cae Madog Cilcain Road, Tafarn Y Gelyn LLanferres, Mold	Project HIGH BARN, BIRCH VALE, HIGH PEAK				Job no. GMN240	
	Calcs for SECONDARY MEMBERS DESIGN CHECKS				Start page no./Revision 1	
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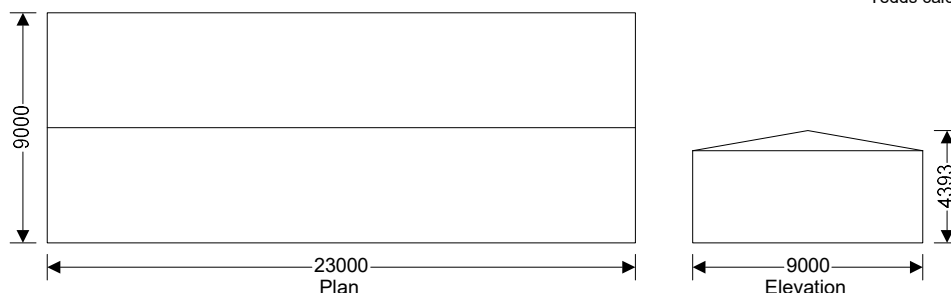
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: WIND LOADING (BS6399)

In accordance with BS6399

Tedds calculation version 3.0.19



Building data

Type of roof;	Duopitch
Length of building;	L = 23000 mm
Width of building;	W = 9000 mm
Height to eaves;	H = 3600 mm
Pitch of roof;	$\alpha_0 = 10.0$ deg
Reference height;	H _r = 4393 mm

Dynamic classification

Building type factor (Table 1);	K _b = 1.0
Dynamic augmentation factor (1.6.1);	C _r = [K _b × (H _r / (0.1 m)) ^{0.75}] / (800 × log(H _r / (0.1 m))) = 0.01

Site wind speed

Location;	Manchester
Basic wind speed (Figure 6 BS6399:Pt 2)	V _b = 22.3 m/s
Site altitude	Δ _s = 175 m
Upwind distance from sea to site	d _{sea} = 40 km
Direction factor	S _d = 1.00
Seasonal factor	S _s = 1.00
Probability factor	S _p = 1.00
Critical gap between buildings;	g = 5000 mm
Topography not significant	
Altitude factor;	S _a = 1 + 0.001 × Δ _s / 1m = 1.18
Site wind speed	V _s = V _b × S _a × S _d × S _s × S _p = 26.2 m/s
Terrain category;	Country
Displacement height (sheltering effect excluded);	H _d = 0mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

Dynamic pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought);	H _{ref} = 3600 mm
Effective height;	H _e = max(H _{ref} - H _d , 0.4 × H _{ref}) = 3600 mm
Fetch factor (Table 22);	S _c = 0.844

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Turbulence factor (Table 22);

$$S_t = \mathbf{0.203}$$

Gust peak factor;

$$g_t = \mathbf{3.44}$$

Terrain and building factor;

$$S_b = S_c \times (1 + (g_t \times S_t) + S_h) = \mathbf{1.43}$$

Effective wind speed;

$$V_e = V_s \times S_b = \mathbf{37.5 \text{ m/s}}$$

Dynamic pressure;

$$q_s = 0.613 \text{ kg/m}^3 \times V_e^2 = \mathbf{0.864 \text{ kN/m}^2}$$

Dynamic pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought);

$$H_{ref} = \mathbf{4393 \text{ mm}}$$

Effective height;

$$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref}) = \mathbf{4393 \text{ mm}}$$

Fetch factor (Table 22);

$$S_c = \mathbf{0.888}$$

Turbulence factor (Table 22);

$$S_t = \mathbf{0.197}$$

Gust peak factor;

$$g_t = \mathbf{3.44}$$

Terrain and building factor;

$$S_b = S_c \times (1 + (g_t \times S_t) + S_h) = \mathbf{1.49}$$

Effective wind speed;

$$V_e = V_s \times S_b = \mathbf{39.0 \text{ m/s}}$$

Dynamic pressure;

$$q_s = 0.613 \text{ kg/m}^3 \times V_e^2 = \mathbf{0.933 \text{ kN/m}^2}$$

Size effect factors

Diagonal dimension for gablewall;

$$a_{eg} = \mathbf{9.7 \text{ m}}$$

External size effect factor gablewall;

$$C_{aeg} = \mathbf{0.950}$$

Diagonal dimension for side wall;

$$a_{es} = \mathbf{23.3 \text{ m}}$$

External size effect factor side wall;

$$C_{aes} = \mathbf{0.884}$$

Diagonal dimension for roof;

$$a_{er} = \mathbf{23.5 \text{ m}}$$

External size effect factor roof;

$$C_{aer} = \mathbf{0.883}$$

Room/storey volume for internal size effect factor;

$$V_i = \mathbf{0.125 \text{ m}^3}$$

Diagonal dimension for internal size effect factors;

$$a_i = 10 \times (V_i)^{1/3} = \mathbf{5.000 \text{ m}}$$

Internal size effect factor;

$$C_{ai} = \mathbf{1.000}$$

Pressures and forces

Net pressure;

$$p = q_s \times C_{pe} \times C_{ae} - q_s \times C_{pi} \times C_{ai};$$

Net force;

$$F_w = p \times A_{ref};$$

Roof load case 1 - Wind 0, C_{pi} 0.20, $-C_{pe}$

Zone	Ext pressure coefficient, C_{pe}	Dynamic pressure, q_s (kN/m ²)	External size factor, C_{ae}	Net Pressure, p (kN/m ²)	Area, A_{ref} (m ²)	Net force, F_w (kN)
A (-ve)	-1.45	0.93	0.883	-1.38	7.84	-10.83
B (-ve)	-1.00	0.93	0.883	-1.01	12.68	-12.81
C (-ve)	-0.50	0.93	0.883	-0.60	84.57	-50.61
E (-ve)	-1.10	0.93	0.883	-1.09	7.84	-8.57
F (-ve)	-0.60	0.93	0.883	-0.68	12.68	-8.63
G (-ve)	-0.45	0.93	0.883	-0.56	84.57	-47.12

Total vertical net force;

$$F_{w,v} = \mathbf{-136.46 \text{ kN}}$$

Total horizontal net force;

$$F_{w,h} = \mathbf{-1.72 \text{ kN}}$$

Walls load case 1 - Wind 0, C_{pi} 0.20, $-C_{pe}$

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Zone	Ext pressure coefficient, C_{pe}	Dynamic pressure, q_s (kN/m ²)	External size factor, C_{ae}	Net Pressure, p (kN/m ²)	Area, A_{ref} (m ²)	Net force, F_w (kN)
A	-1.56	0.93	0.950	-1.57	6.60	-10.34
B	-0.89	0.93	0.950	-0.97	28.60	-27.79
C	-0.84	0.93	0.950	-0.93	0.77	-0.72
w	0.73	0.86	0.884	0.38	82.80	31.53
I	-0.50	0.86	0.884	-0.55	82.80	-45.91

Overall loading

Equiv leeward net force for overall section;

$$F_I = F_{w,wi} = -45.9 \text{ kN}$$

Net windward force for overall section;

$$F_w = F_{w,ww} = 31.5 \text{ kN}$$

Overall loading overall section;

$$F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_I + F_{w,h}) = 65.2 \text{ kN}$$

Roof load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient, C_{pe}	Dynamic pressure, q_s (kN/m ²)	External size factor, C_{ae}	Net Pressure, p (kN/m ²)	Area, A_{ref} (m ²)	Net force, F_w (kN)
A (+ve)	0.10	0.93	0.883	0.36	7.84	2.84
B (+ve)	0.10	0.93	0.883	0.36	12.68	4.59
C (+ve)	0.10	0.93	0.883	0.36	84.57	30.63
E (+ve)	-1.10	0.93	0.883	-0.63	7.84	-4.91
F (+ve)	-0.60	0.93	0.883	-0.21	12.68	-2.72
G (+ve)	-0.45	0.93	0.883	-0.09	84.57	-7.69

Total vertical net force;

$$F_{w,v} = 22.39 \text{ kN}$$

Total horizontal net force;

$$F_{w,h} = 9.27 \text{ kN}$$

Walls load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient, C_{pe}	Dynamic pressure, q_s (kN/m ²)	External size factor, C_{ae}	Net Pressure, p (kN/m ²)	Area, A_{ref} (m ²)	Net force, F_w (kN)
A	-1.56	0.93	0.950	-1.10	6.60	-7.27
B	-0.89	0.93	0.950	-0.51	28.60	-14.45
C	-0.84	0.93	0.950	-0.47	0.77	-0.36
w	0.73	0.86	0.884	0.81	82.80	67.29
I	-0.50	0.86	0.884	-0.12	82.80	-10.15

Overall loading

Equiv leeward net force for overall section;

$$F_I = F_{w,wi} = -10.2 \text{ kN}$$

Net windward force for overall section;

$$F_w = F_{w,ww} = 67.3 \text{ kN}$$

Overall loading overall section;

$$F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_I + F_{w,h}) = 74.7 \text{ kN}$$

Roof load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, C_{pe}	Dynamic pressure, q_s (kN/m ²)	External size factor, C_{ae}	Net Pressure, p (kN/m ²)	Area, A_{ref} (m ²)	Net force, F_w (kN)
A (-ve)	-1.80	0.93	0.883	-1.67	4.02	-6.70
B (-ve)	-1.30	0.93	0.883	-1.26	4.02	-5.05
C (-ve)	-0.60	0.93	0.883	-0.68	32.12	-21.87
D (-ve)	-0.45	0.93	0.883	-0.56	170.04	-94.75

Total vertical net force;

$$F_{w,v} = -126.41 \text{ kN}$$

Total horizontal net force;

$$F_{w,h} = 0.00 \text{ kN}$$

Walls load case 3 - Wind 90, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, C_{pe}	Dynamic pressure, q_s (kN/m ²)	External size factor, C_{ae}	Net Pressure, p (kN/m ²)	Area, A_{ref} (m ²)	Net force, F_w (kN)
A	-1.48	0.86	0.884	-1.31	5.18	-6.77
B	-0.86	0.86	0.884	-0.83	20.74	-17.22
C	-0.74	0.86	0.884	-0.74	56.88	-42.16
w	0.60	0.93	0.950	0.35	35.97	12.41
I	-0.50	0.93	0.950	-0.63	35.97	-22.64

Overall loading

Equiv leeward net force for overall section;

$$F_I = F_{w,wi} = -22.6 \text{ kN}$$

Net windward force for overall section;

$$F_w = F_{w,ww} = 12.4 \text{ kN}$$

Overall loading overall section;

$$F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_I + F_{w,h}) = 30.2 \text{ kN}$$

Roof load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

Zone	Ext pressure coefficient, C_{pe}	Dynamic pressure, q_s (kN/m ²)	External size factor, C_{ae}	Net Pressure, p (kN/m ²)	Area, A_{ref} (m ²)	Net force, F_w (kN)
A (-ve)	-1.80	0.93	0.883	-1.20	4.02	-4.83
B (-ve)	-1.30	0.93	0.883	-0.79	4.02	-3.18
C (-ve)	-0.60	0.93	0.883	-0.21	32.12	-6.89
D (-ve)	-0.45	0.93	0.883	-0.09	170.04	-15.46

Total vertical net force;

$$F_{w,v} = -29.89 \text{ kN}$$

Total horizontal net force;

$$F_{w,h} = 0.00 \text{ kN}$$

Walls load case 4 - Wind 90, c_{pi} -0.3, $-c_{pe}$

Zone	Ext pressure coefficient, C_{pe}	Dynamic pressure, q_s (kN/m ²)	External size factor, C_{ae}	Net Pressure, p (kN/m ²)	Area, A_{ref} (m ²)	Net force, F_w (kN)
A	-1.48	0.86	0.884	-0.87	5.18	-4.53
B	-0.86	0.86	0.884	-0.40	20.74	-8.26
C	-0.74	0.86	0.884	-0.31	56.88	-17.59
w	0.60	0.93	0.950	0.81	35.97	29.18
I	-0.50	0.93	0.950	-0.16	35.97	-5.87

Overall loading

Equiv leeward net force for overall section;

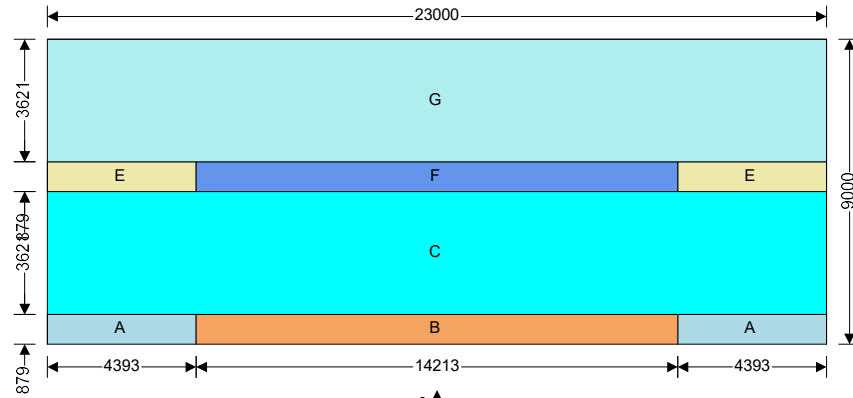
$$F_I = F_{w,wi} = -5.9 \text{ kN}$$

Net windward force for overall section;

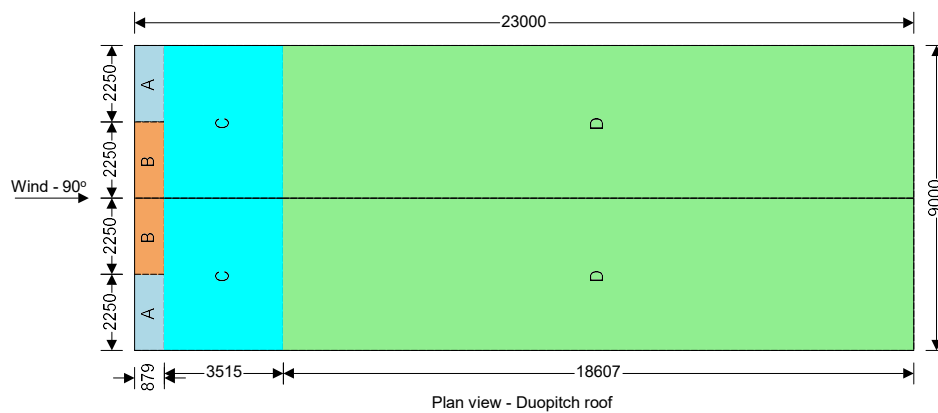
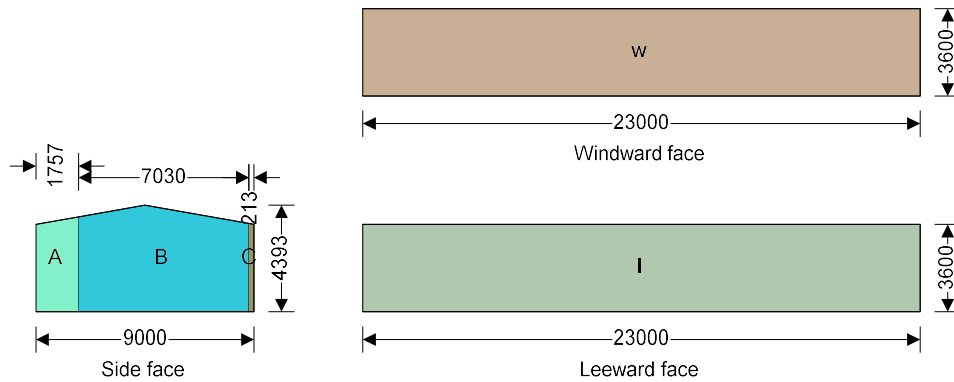
$$F_w = F_{w,ww} = 29.2 \text{ kN}$$

Overall loading overall section;

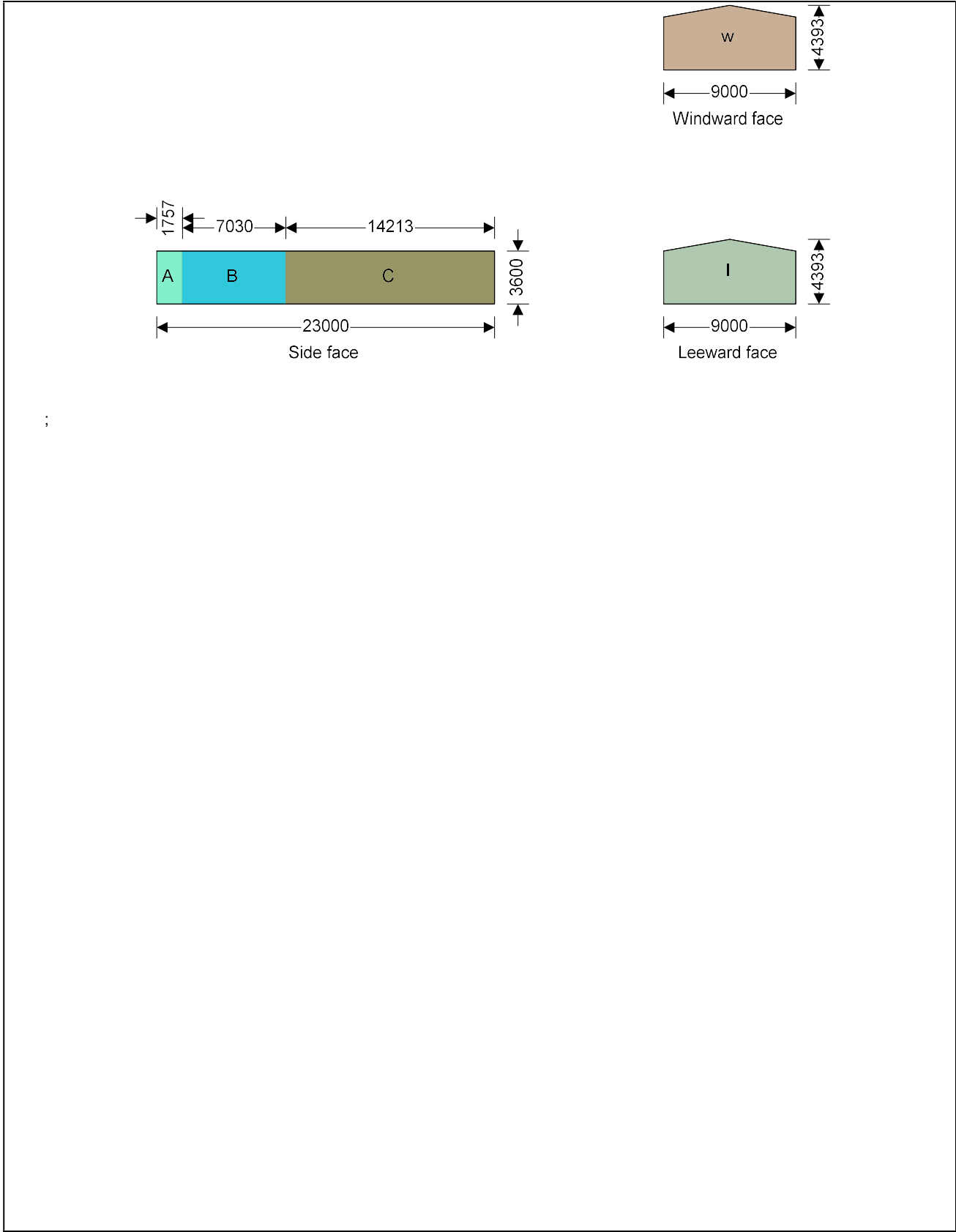
$$F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h}) = 30.2 \text{ kN}$$



Wind - 0°
Plan view - Duopitch roof

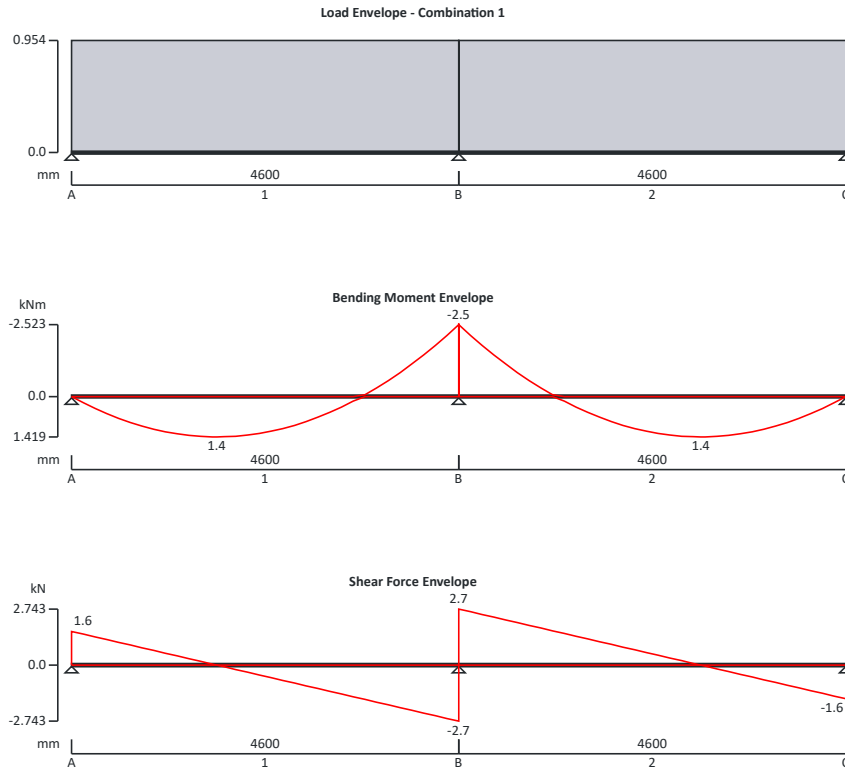


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TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03



Applied loading

Beam loads

Dead self weight of beam $\times 1$

Dead full UDL 0.900 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.00$

Imposed $\times 1.00$

Span 1

Dead $\times 1.00$

Imposed $\times 1.00$

Support B

Dead $\times 1.00$

Imposed $\times 1.00$

Span 2

Dead $\times 1.00$

Imposed $\times 1.00$

Support C

Dead $\times 1.00$

Imposed $\times 1.00$

Analysis results

Maximum moment;

 $M_{\max} = 1.419 \text{ kNm};$
 $M_{\min} = -2.523 \text{ kNm}$

Design moment;

 $M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 2.523 \text{ kNm}$

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Maximum shear;

$$F_{\max} = 2.743 \text{ kN};$$

$$F_{\min} = -2.743 \text{ kN}$$

Design shear;

$$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 2.743 \text{ kN}$$

Total load on beam;

$$W_{\text{tot}} = 8.777 \text{ kN}$$

Reactions at support A;

$$R_{A_{\max}} = 1.646 \text{ kN};$$

$$R_{A_{\min}} = 1.646 \text{ kN}$$

Unfactored dead load reaction at support A;

$$R_{A_{\text{Dead}}} = 1.646 \text{ kN}$$

Reactions at support B;

$$R_{B_{\max}} = 5.486 \text{ kN};$$

$$R_{B_{\min}} = 5.486 \text{ kN}$$

Unfactored dead load reaction at support B;

$$R_{B_{\text{Dead}}} = 5.486 \text{ kN}$$

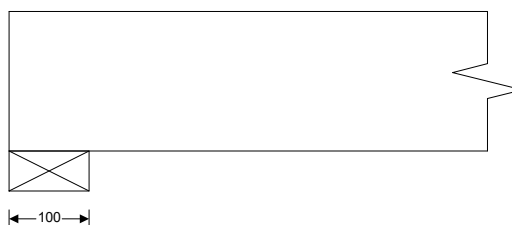
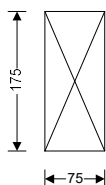
Reactions at support C;

$$R_{C_{\max}} = 1.646 \text{ kN};$$

$$R_{C_{\min}} = 1.646 \text{ kN}$$

Unfactored dead load reaction at support C;

$$R_{C_{\text{Dead}}} = 1.646 \text{ kN}$$



Timber section details

Breadth of sections;

$$b = 75 \text{ mm}$$

Depth of sections;

$$h = 175 \text{ mm}$$

Number of sections in member;

$$N = 1$$

Overall breadth of member;

$$b_b = N \times b = 75 \text{ mm}$$

Timber strength class;

C24

Member details

Service class of timber;

1

Load duration;

Long term

Length of span 1;

$$L_{s1} = 4600 \text{ mm}$$

Length of span 2;

$$L_{s2} = 4600 \text{ mm}$$

Length of bearing;

$$L_b = 100 \text{ mm}$$

Section properties

Cross sectional area of member;

$$A = N \times b \times h = 13125 \text{ mm}^2$$

Section modulus;

$$Z_x = N \times b \times h^2 / 6 = 382813 \text{ mm}^3$$

$$Z_y = h \times (N \times b)^2 / 6 = 164063 \text{ mm}^3$$

Second moment of area;

$$I_x = N \times b \times h^3 / 12 = 33496094 \text{ mm}^4$$

$$I_y = h \times (N \times b)^3 / 12 = 6152344 \text{ mm}^4$$

Radius of gyration;

$$i_x = \sqrt{I_x / A} = 50.5 \text{ mm}$$

$$i_y = \sqrt{I_y / A} = 21.7 \text{ mm}$$

Modification factors

Duration of loading - Table 17;

$$K_3 = 1.00$$

Bearing stress - Table 18;

$$K_4 = 1.10$$

Total depth of member - cl.2.10.6;

$$K_7 = (300 \text{ mm} / h)^{0.11} = 1.06$$

Load sharing - cl.2.9;

$$K_8 = 1.00$$

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by direct connection of sheathing, deck or joists

Permissible depth-to-breadth ratio - Table 19;

$$5.00$$

Actual depth-to-breadth ratio;

$$h / (N \times b) = 2.33$$

PASS - Lateral support is adequate

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Compression perpendicular to grain

Permissible bearing stress (no wane);

$$\sigma_{c_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = \mathbf{2.640 \text{ N/mm}^2}$$

Applied bearing stress;

$$\sigma_{c_a} = R_{B_max} / (N \times b \times L_b) = \mathbf{0.731 \text{ N/mm}^2}$$

$$\sigma_{c_a} / \sigma_{c_adm} = \mathbf{0.277}$$

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

Bending parallel to grain

Permissible bending stress;

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{7.958 \text{ N/mm}^2}$$

Applied bending stress;

$$\sigma_{m_a} = M / Z_x = \mathbf{6.592 \text{ N/mm}^2}$$

$$\sigma_{m_a} / \sigma_{m_adm} = \mathbf{0.828}$$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress;

$$\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.710 \text{ N/mm}^2}$$

Applied shear stress;

$$\tau_a = 3 \times F / (2 \times A) = \mathbf{0.313 \text{ N/mm}^2}$$

$$\tau_a / \tau_{adm} = \mathbf{0.442}$$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection;

$$E = E_{min} = \mathbf{7200 \text{ N/mm}^2}$$

Permissible deflection;

$$\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) = \mathbf{13.800 \text{ mm}}$$

Bending deflection;

$$\delta_{b_s1} = \mathbf{9.593 \text{ mm}}$$

Shear deflection;

$$\delta_{v_s1} = \mathbf{0.288 \text{ mm}}$$

Total deflection;

$$\delta_a = \delta_{b_s1} + \delta_{v_s1} = \mathbf{9.882 \text{ mm}}$$

$$\delta_a / \delta_{adm} = \mathbf{0.716}$$

PASS - Total deflection is less than permissible deflection

;

MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005

Tedds calculation version 1.2.16

Masonry panel details

Unreinforced masonry wall without openings

Panel length;

$L = 4500$ mm

Panel height;

$h = 1800$ mm

Panel support conditions

;

All edges supported, bottom, right and left continuous

Effective panel length;

$L_{ef} = 1.0 \times L = 4500$ mm

Effective panel height;

$h_{ef} = 1.0 \times h = 1800$ mm



Single-leaf wall construction details

Wall thickness;

$t = 140$ mm

Effective wall thickness;

$t_{ef} = t = 140$ mm



Masonry details

Masonry type;

Aggregate concrete blocks with no voids

Compressive strength of unit;

$p_{unit} = 7.3$ N/mm²

Mortar strength Class/Designation;

M6 / (ii);

Height of masonry units;

$h_b = 215$ mm

Density of masonry;

$\gamma = 18.0$ kN/m³

Least horizontal dimension of masonry units;

$t_{min} = 140$ mm

Ratio of height to least horizontal dimension;

$h_b / t_{min} = 1.54$

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From BS5628-1 Table 2c & d - Characteristic compressive strength of masonry

Characteristic compressive strength; $f_k = 5.34 \text{ N/mm}^2$

From BS5628-1 Table 3 - Characteristic flexural strength of masonry

Plane of failure parallel to bed joints; $f_{kx_para} = 0.22 \text{ N/mm}^2$

Plane of failure perpendicular to bed joints; $f_{kx_perp} = 0.53 \text{ N/mm}^2$

Lateral loading details

Characteristic wind load on panel; $W_k = 1.000 \text{ kN/m}^2$

Partial safety factors for material strength

Category of manufacturing control; **Category II**

Category of construction control; **Normal**

Partial safety factor for masonry in compression; $\gamma_{mc} = 3.50$

Partial safety factor for masonry in flexure; $\gamma_{mf} = 3.00$

Partial safety factor for masonry in shear; $\gamma_{mv} = 2.50$

Horizontal loading (cl 32)

Limiting dimensions (cl 32.3)

Area of panel; $A_p = h \times L = 8.1 \text{ m}^2$

Limiting area of panel; $A_{max} = 2025 \times t_{ef}^2 = 39.7 \text{ m}^2$

PASS - Area of panel does not exceed limiting area of panel

Limiting panel dimension; $L_{max} = 50 \times t_{ef} = 7000 \text{ mm}$

PASS - Limiting panel dimension is not exceeded

Partial safety factors for design loads

Partial safety factor for design wind load; $\gamma_{fW} = 1.40$

Partial safety factor for design dead load; $\gamma_{fG} = 0.90$

Design moments of resistance in panels (cl 32.4.2)

Design vertical compressive stress; $g_d = \gamma_{fG} \times G_k / t = 0.00 \text{ N/mm}^2$

Enhanced flexural strength of masonry; $f_{ka_para} = f_{kx_para} + \gamma_{mf} \times g_d = 0.22 \text{ N/mm}^2$

Section modulus of wall; $Z = t^2 / 6 = 3266667 \text{ mm}^3/\text{m}$

Elastic design moment of resistance; $M_d = f_{kx_perp} \times Z / \gamma_{mf} = 0.581 \text{ kNm/m}$

Design moment in panels (cl 32.4.2)

Orthogonal strength ratio; $\mu = f_{ka_para} / f_{kx_perp} = 0.42$

Using yield line analysis to calculate bending moment coefficient

Bending moment coefficient; $\alpha = 0.015$

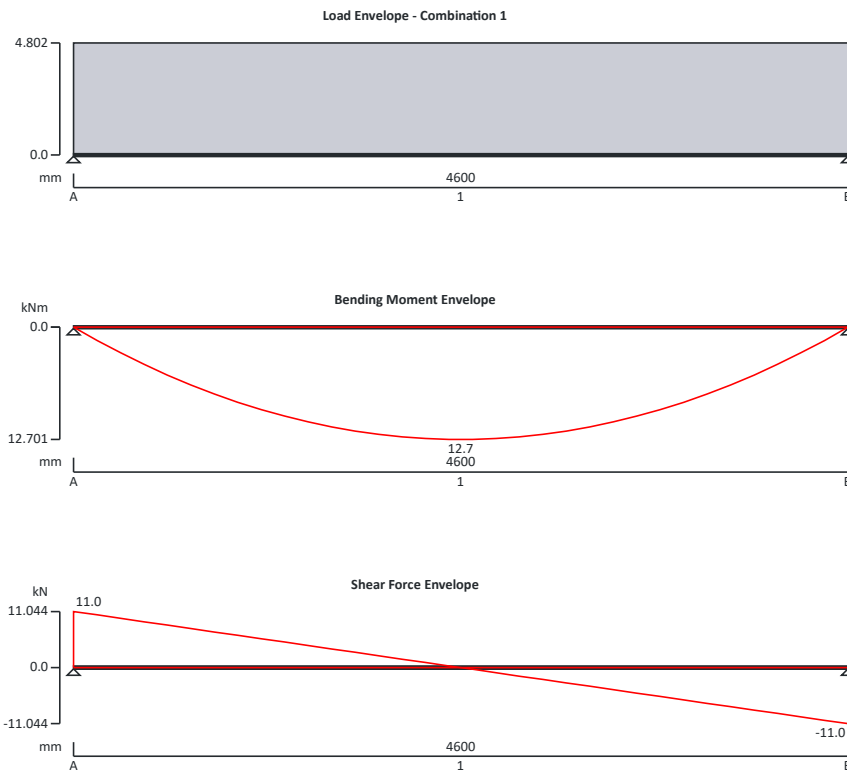
Design moment in wall; $M = \alpha \times W_k \times \gamma_{fW} \times L^2 = 0.412 \text{ kNm/m}$

PASS - Resistance moment exceeds design moment

: STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.08



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Wind full UDL 4.5 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Wind $\times 1.00$

Dead $\times 1.40$

Imposed $\times 1.60$

Wind $\times 1.00$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$

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Wind $\times 1.00$

Analysis results

Maximum moment;

$M_{max} = 12.7 \text{ kNm};$

$M_{min} = 0 \text{ kNm}$

Maximum shear;

$V_{max} = 11 \text{ kN};$

$V_{min} = -11 \text{ kN}$

Deflection;

$\delta_{max} = 4.5 \text{ mm};$

$\delta_{min} = 0 \text{ mm}$

Maximum reaction at support A;

$R_{A_{max}} = 11 \text{ kN};$

$R_{A_{min}} = 11 \text{ kN}$

Unfactored dead load reaction at support A;

$R_{A_{Dead}} = 0.5 \text{ kN}$

Unfactored wind load reaction at support A;

$R_{A_{Wind}} = 10.4 \text{ kN}$

Maximum reaction at support B;

$R_{B_{max}} = 11 \text{ kN};$

$R_{B_{min}} = 11 \text{ kN}$

Unfactored dead load reaction at support B;

$R_{B_{Dead}} = 0.5 \text{ kN}$

Unfactored wind load reaction at support B;

$R_{B_{Wind}} = 10.4 \text{ kN}$

Section details

Section type;

UKB 254x102x22 (Tata Steel Advance)

Steel grade;

S275

From table 9: Design strength p_y

Thickness of element;

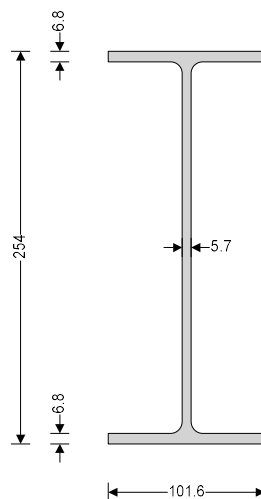
$\max(T, t) = 6.8 \text{ mm}$

Design strength;

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

Modulus of elasticity;

$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

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

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

Internal compression parts - Table 11

Depth of section;

$d = 225.2 \text{ mm}$

$d / t = 39.5 \times \epsilon \leq 80 \times \epsilon;$

Class 1 plastic

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Outstand flanges - Table 11

Width of section;

$$b = B / 2 = 50.8 \text{ mm}$$

$$b / T = 7.5 \times \varepsilon \leq 9 \times \varepsilon;$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force;

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 11 \text{ kN}$$

$$d / t < 70 \times \varepsilon$$

Web does not need to be checked for shear buckling

Shear area;

$$A_v = t \times D = 1448 \text{ mm}^2$$

Design shear resistance;

$$P_v = 0.6 \times p_y \times A_v = 238.9 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment;

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 12.7 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2;

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 71.2 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed and wind loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 360 = 12.778 \text{ mm}$$

Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 4.505 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

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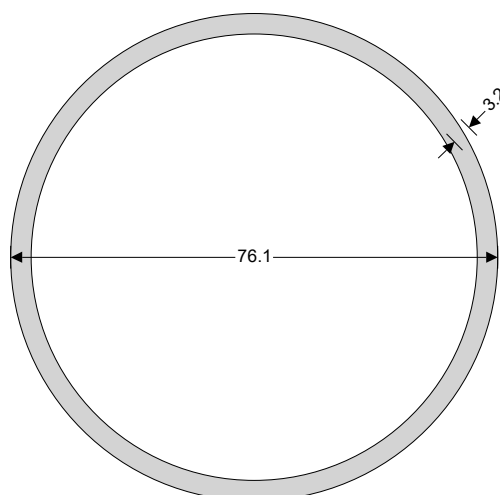
: STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.08

Section details

Section type;	CHS 76.1x3.2 (Tata Steel Celsius (Gr355 Gr420 Gr460))
Steel grade;	S275
From table 9: Design strength p_y	
Thickness of element;	$t = 3.2 \text{ mm}$
Design strength;	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity;	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Distance between major axis restraints;	$L_x = 6350 \text{ mm}$
Distance between minor axis restraints;	$L_y = 6350 \text{ mm}$

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

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

Tubular sections - Table 12

$$D / t = 23.8 \times \varepsilon \leq 80 \times \varepsilon^2;$$

Class 3 semi-compact

Section is class 3 semi-compact

Compression members - Section 4.7

Design compression force;	$F_c = 15 \text{ kN}$
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Effective length for major (x-x) axis buckling - Section 4.7.3

Effective length for buckling;	$L_{Ex} = L_x \times K_x = 6350 \text{ mm}$
Slenderness ratio - cl.4.7.2;	$\lambda_{xx} = L_{Ex} / r_{xx} = 246.135$

Compressive strength - Section 4.7.5

Limiting slenderness;	$\lambda_{0.2} = 0.2 \times (\pi^2 \times E / p_y)^{0.5} = 17.155$
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Strut curve - Table 23;

Robertson constant;

Perry factor;

Euler stress;

a

$$\alpha_x = 2.0$$

$$\eta_x = \alpha_x \times (\lambda_x - \lambda_0) / 1000 = 0.458$$

$$p_{Ex} = \pi^2 \times E / \lambda_x^2 = 33.4 \text{ N/mm}^2$$

$$\phi_x = (p_y + (\eta_x + 1) \times p_{Ex}) / 2 = 161.8 \text{ N/mm}^2$$

$$p_{cx} = p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5}) = 31.4 \text{ N/mm}^2$$

Compressive strength - Annex C.1;

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4;

$$P_{cx} = A \times p_{cx} = 23 \text{ kN}$$

PASS - Compression resistance exceeds design compression force

Effective length for minor (y-y) axis buckling - Section 4.7.3

Effective length for buckling;

$$L_{Ey} = L_y \times K_y = 6350 \text{ mm}$$

Slenderness ratio - cl.4.7.2;

$$\lambda_y = L_{Ey} / r_{yy} = 246.135$$

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;

$$\alpha_y = 2.0$$

Perry factor;

$$\eta_y = \alpha_y \times (\lambda_y - \lambda_0) / 1000 = 0.458$$

Euler stress;

$$p_{Ey} = \pi^2 \times E / \lambda_y^2 = 33.4 \text{ N/mm}^2$$

$$\phi_y = (p_y + (\eta_y + 1) \times p_{Ey}) / 2 = 161.8 \text{ N/mm}^2$$

Compressive strength - Annex C.1;

$$p_{cy} = p_{Ey} \times p_y / (\phi_y + (\phi_y^2 - p_{Ey} \times p_y)^{0.5}) = 31.4 \text{ N/mm}^2$$

Compression resistance - Section 4.7.4

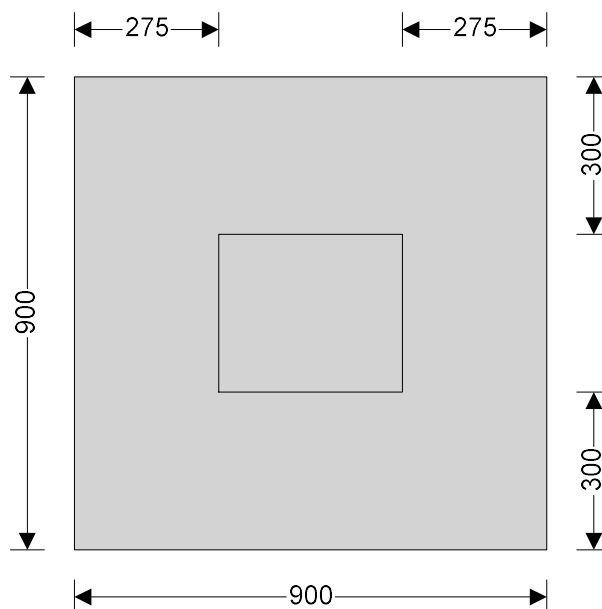
Compression resistance - cl.4.7.4;

$$P_{cy} = A \times p_{cy} = 23 \text{ kN}$$

PASS - Compression resistance exceeds design compression force

PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.08



Pad footing details

Length of pad footing;

$L = 900$ mm

Width of pad footing;

$B = 900$ mm

Area of pad footing;

$A = L \times B = 0.810$ m²

Depth of pad footing;

$h = 600$ mm

Depth of soil over pad footing;

$h_{\text{soil}} = 200$ mm

Density of concrete;

$\rho_{\text{conc}} = 23.6$ kN/m³

Column details

Column base length;

$l_A = 350$ mm

Column base width;

$b_A = 300$ mm

Column eccentricity in x;

$e_{Px} = 0$ mm

Column eccentricity in y;

$e_{Py} = 0$ mm

Soil details

Density of soil;

$\rho_{\text{soil}} = 20.0$ kN/m³

Design shear strength;

$\phi' = 25.0$ deg

Design base friction;

$\delta = 19.3$ deg

Allowable bearing pressure;

$P_{\text{bearing}} = 250$ kN/m²

Axial loading on column

Dead axial load on column;

$P_{GA} = 5.2$ kN

Imposed axial load on column;

$P_{QA} = 12.4$ kN

Wind axial load on column;

$P_{WA} = 0.0$ kN

Total axial load on column;

$P_A = 17.6$ kN

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Foundation loads

Dead surcharge load;	$F_{Gsur} = 0.000 \text{ kN/m}^2$
Imposed surcharge load;	$F_{Qsur} = 0.000 \text{ kN/m}^2$
Pad footing self weight;	$F_{swt} = h \times \rho_{conc} = 14.160 \text{ kN/m}^2$
Soil self weight;	$F_{soil} = h_{soil} \times \rho_{soil} = 4.000 \text{ kN/m}^2$
Total foundation load;	$F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = 14.7 \text{ kN}$

Horizontal loading on column base

Dead horizontal load in x direction;	$H_{GxA} = 1.3 \text{ kN}$
Imposed horizontal load in x direction;	$H_{QxA} = 4.1 \text{ kN}$
Wind horizontal load in x direction;	$H_{WxA} = 0.0 \text{ kN}$
Total horizontal load in x direction;	$H_{xA} = 5.4 \text{ kN}$
Dead horizontal load in y direction;	$H_{GyA} = 0.0 \text{ kN}$
Imposed horizontal load in y direction;	$H_{QyA} = 0.0 \text{ kN}$
Wind horizontal load in y direction;	$H_{WyA} = 0.0 \text{ kN}$
Total horizontal load in y direction;	$H_{yA} = 0.0 \text{ kN}$

Check stability against sliding

Resistance to sliding due to base friction	$H_{friction} = \max([P_{GA} + (F_{Gsur} + F_{swt} + F_{soil}) \times A], 0 \text{ kN}) \times \tan(\delta) = 7.0 \text{ kN}$
Passive pressure coefficient;	$K_p = (1 + \sin(\phi')) / (1 - \sin(\phi')) = 2.464$

Stability against sliding in x direction

Passive resistance of soil in x direction;	$H_{xpas} = 0.5 \times K_p \times (h^2 + 2 \times h \times h_{soil}) \times B \times \rho_{soil} = 13.3 \text{ kN}$
Total resistance to sliding in x direction;	$H_{xres} = H_{friction} + H_{xpas} = 20.3 \text{ kN}$

PASS - Resistance to sliding is greater than horizontal load in x direction

Check stability against overturning in x direction

Total overturning moment;	$M_{xOT} = M_{xA} + H_{xA} \times h = 3.240 \text{ kNm}$
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Restoring moment in x direction

Foundation loading;	$M_{xsur} = A \times (F_{Gsur} + F_{swt} + F_{soil}) \times L / 2 = 6.619 \text{ kNm}$
Axial loading on column;	$M_{xaxial} = (P_{GA}) \times (L / 2 - e_{PxA}) = 2.340 \text{ kNm}$
Total restoring moment;	$M_{xres} = M_{xsur} + M_{xaxial} = 8.959 \text{ kNm}$

PASS - Overturning safety factor exceeds the minimum of 1.5 in the x direction

Calculate pad base reaction

Total base reaction;	$T = F + P_A = 32.3 \text{ kN}$
Eccentricity of base reaction in x;	$e_{Tx} = (P_A \times e_{PxA} + M_{xA} + H_{xA} \times h) / T = 100 \text{ mm}$
Eccentricity of base reaction in y;	$e_{Ty} = (P_A \times e_{PyA} + M_{yA} + H_{yA} \times h) / T = 0 \text{ mm}$

Check pad base reaction eccentricity

$$\text{abs}(e_{Tx}) / L + \text{abs}(e_{Ty}) / B = 0.111$$

Base reaction acts within middle third of base

Calculate pad base pressures

	$q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = 13.222 \text{ kN/m}^2$
	$q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = 13.222 \text{ kN/m}^2$
	$q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = 66.555 \text{ kN/m}^2$
	$q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = 66.555 \text{ kN/m}^2$
Minimum base pressure;	$q_{min} = \min(q_1, q_2, q_3, q_4) = 13.222 \text{ kN/m}^2$
Maximum base pressure;	$q_{max} = \max(q_1, q_2, q_3, q_4) = 66.555 \text{ kN/m}^2$

PASS - Maximum base pressure is less than allowable bearing pressure

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