30 June 2023 V1.0

Structural Calculations & Specifications

To

Side extension opening

The Coach House,
Sandridge Park,
Sandridge Common,
Melksham,
Wiltshire, SN12 7QU

By

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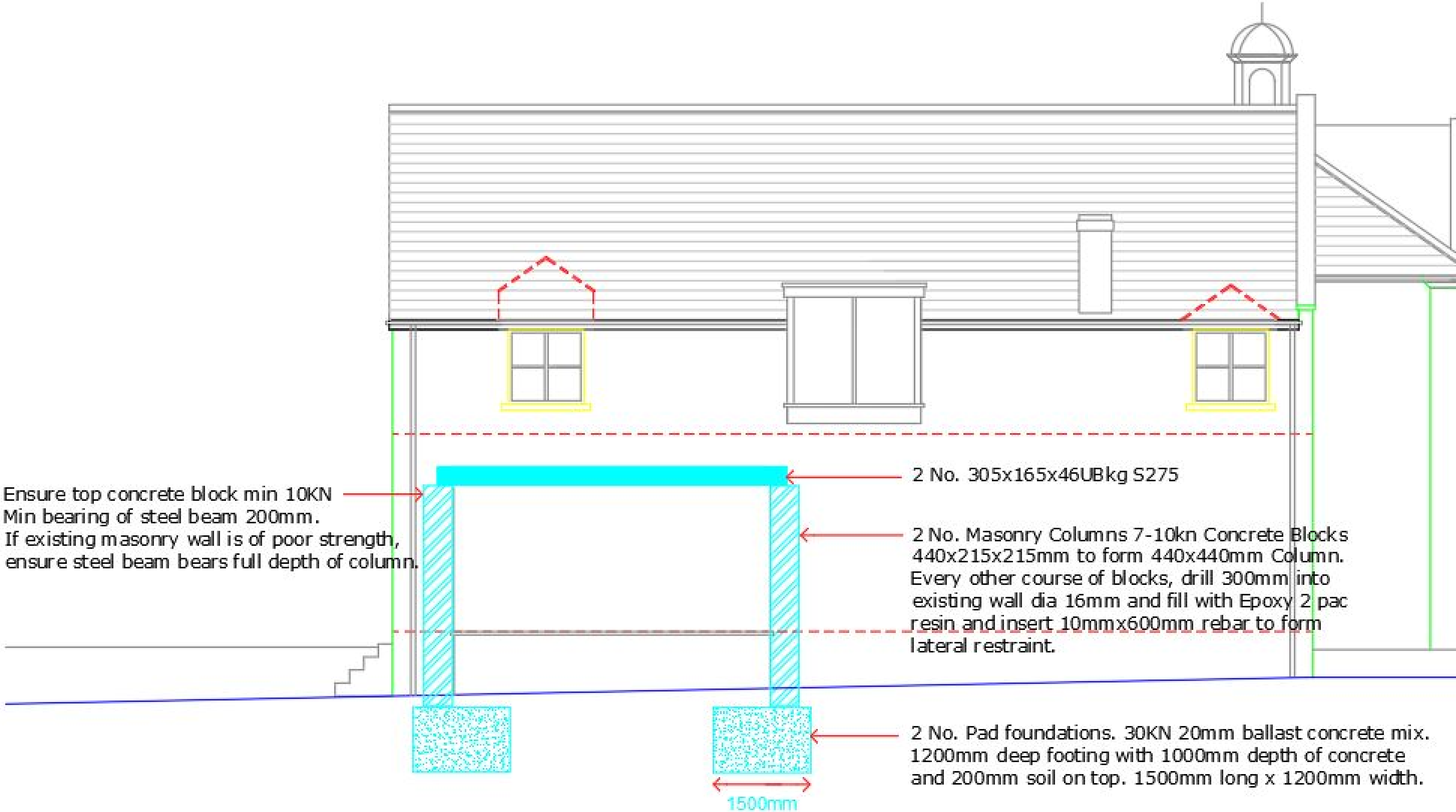


Ensure top concrete block min 10KN
Min bearing of steel beam 200mm.
If existing masonry wall is of poor strength, ensure steel beam bears full depth of column.
2 No. 305x165x46UBkg S275
2 No. Masonry Columns 7-10kn Concrete Blocks 440x215x215mm to form 440x440mm Column. Every other course of blocks, drill 300mm into existing wall dia 16mm and fill with Epoxy 2 pac resin and insert 10mmx600mm rebar to form lateral restraint.
2 No. Pad foundations. 30KN 20mm ballast concrete mix. 1200mm deep footing with 1000mm depth of concrete and 200mm soil on top. 1500mm long x 1200mm width.
1500mm

UNREINFORCED PAD FOUNDATION FOR MASONRY COLUMN

Foundation analysis in accordance with EN1997-1:2004 + A1:2013 incorporating corrigendum February 2009 and the recommended values

Tedds calculation version 3.3.05

Summary table

Description	Unit	Allowable	Actual	Utilisation	Result
Base pressure	kN/m^2	368	171.2	0.465	Pass
Concrete projection	mm	1067	830	0.778	Pass

Pad foundation details

Length of foundation L_x = **1500** mm

Width of foundation L_y = **1200** mm

Foundation area $A = L_x \times L_y$ = **1.800** m^2

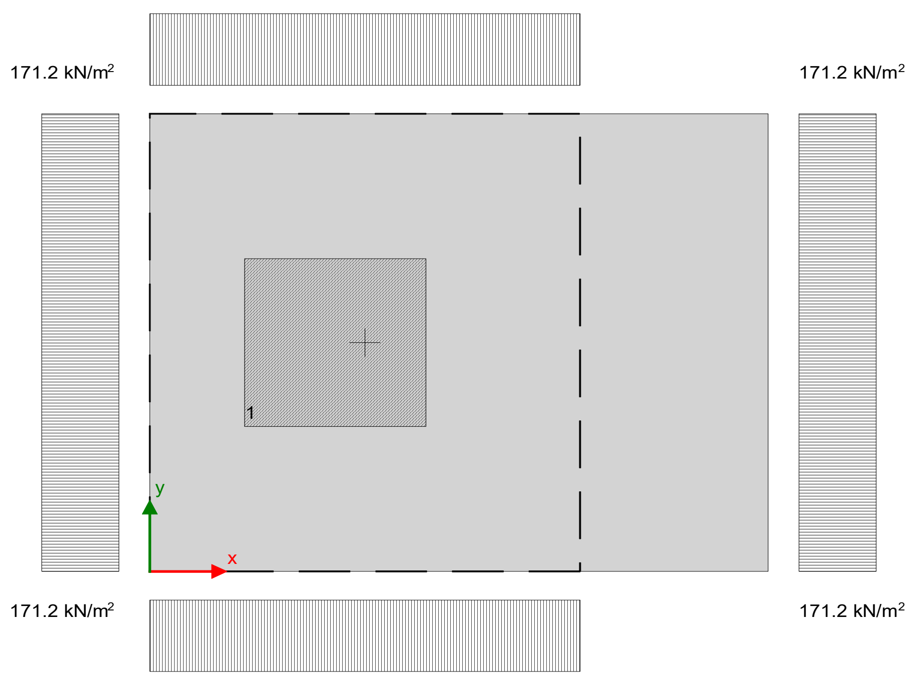
Depth of foundation h = **1000** mm

Depth of soil over foundation h_{soil} = **200** mm

Level of water h_{water} = **0** mm

Density of water γ_{water} = **9.8** kN/m^3

Density of concrete γ_{conc} = **25.0** kN/m^3



Column no.1 details

Length of column l_{x1} = **440** mm

Width of column l_{y1} = **440** mm

position in x-direction x_1 = **450** mm

position in y-direction y_1 = **600** mm

Soil properties

Density of soil	γ_{soil} = **18.0** kN/m^3
Characteristic cohesion	c'_k = **0** kN/m^2
Characteristic effective shear resistance angle	ϕ'_k = **30** deg
Characteristic friction angle	δ_k = **20** deg

Foundation loads

Self weight	$F_{swt} = h \times \gamma_{conc}$ = **25.0** kN/m^2
Soil weight	$F_{soil} = h_{soil} \times \gamma_{soil}$ = **3.6** kN/m^2

Column no.1 loads

Permanent axial load	F_{Gz1} = **150.0** kN
Variable axial load	F_{Qz1} = **10.0** kN

Design approach 1

Partial factors on actions - Combination1

Partial factor set	A1
Permanent unfavourable action - Table A.3	γ_G = **1.35**
Permanent favourable action - Table A.3	γ_{Gf} = **1.00**
Variable unfavourable action - Table A.3	γ_Q = **1.50**
Variable favourable action - Table A.3	γ_{Qf} = **0.00**

Partial factors for soil parameters - Combination1

Soil factor set	M1
Angle of shearing resistance - Table A.4	$\gamma_{\phi'}$ = **1.00**
Effective cohesion - Table A.4	$\gamma_{c'}$ = **1.00**
Weight density - Table A.4	γ_γ = **1.00**

Partial factors for spread foundations - Combination1

Resistance factor set	R1
Bearing - Table A.5	$\gamma_{R.v}$ = **1.00**
Sliding - Table A.5	$\gamma_{R.h}$ = **1.00**

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in z-direction	$F_{dz} = \gamma_G \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_Q \times F_{Qz1}$ = **287.0** kN

Moments on foundation

Moment in x-direction	$M_{dx} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1$ = **150.0** kNm
Moment in y-direction	$M_{dy} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1$ = **172.2** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-direction	$e_x = M_{dx} / F_{dz} - L_x / 2$ = **-227** mm
Eccentricity of base reaction in y-direction	$e_y = M_{dy} / F_{dz} - L_y / 2$ = **0** mm

Effective area of base

Effective length	$L'_x = L_x + 2 \times e_x$ = **1045** mm
Effective width	$L'_y = L_y - 2 \times e_y$ = **1200** mm
Effective area	$A' = L'_x \times L'_y$ = **1.254** m^2

Pad base pressure

Design base pressure — $f_{dz} = F_{dz} / A' = \mathbf{228.8}$ kN/m^2

Ultimate bearing capacity under drained conditions (Annex D.4)

Design angle of shearing resistance — $\phi'_d = \text{atan}(\tan(\phi'_k) / \gamma_{\phi'}) = \mathbf{30.000}$ deg

Design effective cohesion — $c'_d = c'_k / \gamma_{c'} = \mathbf{0.000}$ kN/m^2

Effective overburden pressure — $q = (h + h_{soil}) \times \gamma_{soil} - h_{water} \times \gamma_{water} = \mathbf{21.600}$ kN/m^2

Design effective overburden pressure — $q' = q / \gamma_\gamma = \mathbf{21.600}$ kN/m^2

Bearing resistance factors — $N_q = \text{Exp}(\pi \times \tan(\phi'_d)) \times (\tan(45 \text{ deg} + \phi'_d / 2))^2 = \mathbf{18.401}$

$N_c = (N_q - 1) \times \cot(\phi'_d) = \mathbf{30.140}$

$N_\gamma = 2 \times (N_q - 1) \times \tan(\phi'_d) = \mathbf{20.093}$

Foundation shape factors — $s_q = 1 + (L'_x / L'_y) \times \sin(\phi'_d) = \mathbf{1.436}$

$s_\gamma = 1 - 0.3 \times (L'_x / L'_y) = \mathbf{0.739}$

$s_c = (s_q \times N_q - 1) / (N_q - 1) = \mathbf{1.461}$

Load inclination factors — $H = \mathbf{0.0}$ kN

$m_y = [2 + (L'_y / L'_x)] / [1 + (L'_y / L'_x)] = \mathbf{1.466}$

$m_x = [2 + (L'_x / L'_y)] / [1 + (L'_x / L'_y)] = \mathbf{1.534}$

$m = m_x = \mathbf{1.534}$

$i_q = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^m = \mathbf{1.000}$

$i_\gamma = [1 - H / (F_{dz} + A' \times c'_d \times \cot(\phi'_d))]^{m+1} = \mathbf{1.000}$

$i_c = i_q - (1 - i_q) / (N_c \times \tan(\phi'_d)) = \mathbf{1.000}$

Ultimate bearing capacity — $n_f = c'_d \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times \gamma_{soil} \times L'_x \times N_\gamma \times s_\gamma \times i_\gamma = \mathbf{710.2}$ kN/m^2

PASS - Ultimate bearing capacity exceeds design base pressure

Design approach 1

Partial factors on actions - Combination2

Partial factor set	A2
Permanent unfavourable action - Table A.3	$\gamma_G = \mathbf{1.00}$
Permanent favourable action - Table A.3	$\gamma_{Gf} = \mathbf{1.00}$
Variable unfavourable action - Table A.3	$\gamma_Q = \mathbf{1.30}$
Variable favourable action - Table A.3	$\gamma_{Qf} = \mathbf{0.00}$

Partial factors for soil parameters - Combination2

Soil factor set	M2
Angle of shearing resistance - Table A.4	$\gamma_{\phi'} = \mathbf{1.25}$
Effective cohesion - Table A.4	$\gamma_{c'} = \mathbf{1.25}$
Weight density - Table A.4	$\gamma_\gamma = \mathbf{1.00}$

Partial factors for spread foundations - Combination2

Resistance factor set	R1
Bearing - Table A.5	$\gamma_{R.v} = \mathbf{1.00}$
Sliding - Table A.5	$\gamma_{R.h} = \mathbf{1.00}$

Bearing resistance (Section 6.5.2)

Forces on foundation

Force in z-direction — $F_{dz} = \gamma_G \times (A \times (F_{swt} + F_{soil}) + F_{Gz1}) + \gamma_Q \times F_{Qz1} = \mathbf{214.5}$ kN

Moments on foundation

Moment in x-direction — $M_{dx} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_x / 2 + F_{Gz1} \times x_1) + \gamma_Q \times F_{Qz1} \times x_1$ = **112.0** kNm

Moment in y-direction — $M_{dy} = \gamma_G \times (A \times (F_{swt} + F_{soil}) \times L_y / 2 + F_{Gz1} \times y_1) + \gamma_Q \times F_{Qz1} \times y_1$ = **128.7** kNm

Eccentricity of base reaction

Eccentricity of base reaction in x-direction — $e_x = M_{dx} / F_{dz} - L_x / 2$ = **-228** mm

Eccentricity of base reaction in y-direction — $e_y = M_{dy} / F_{dz} - L_y / 2$ = **0** mm

Effective area of base

Effective length — $L'_x = L_x + 2 \times e_x$ = **1044** mm

Effective width — $L'_y = L_y - 2 \times e_y$ = **1200** mm

Effective area — $A' = L'_x \times L'_y$ = **1.253** m^2

Pad base pressure

Design base pressure — $f_{dz} = F_{dz} / A'$ = **171.2** kN/m^2

Ultimate bearing capacity under drained conditions (Annex D.4)

Design angle of shearing resistance — $\phi'_d = atan(tan(\phi'_k) / \gamma_{\phi'})$ = **24.791** deg

Design effective cohesion — $c'_d = c'_k / \gamma_{c'}$ = **0.000** kN/m^2

Effective overburden pressure — $q = (h + h_{soil}) \times \gamma_{soil} - h_{water} \times \gamma_{water}$ = **21.600** kN/m^2

Design effective overburden pressure — $q' = q / \gamma_\gamma$ = **21.600** kN/m^2

Bearing resistance factors — $N_q = Exp(\pi \times tan(\phi'_d)) \times (tan(45 deg + \phi'_d / 2))^2$ = **10.431**

$N_c = (N_q - 1) \times cot(\phi'_d)$ = **20.418**

$N_\gamma = 2 \times (N_q - 1) \times tan(\phi'_d)$ = **8.712**

Foundation shape factors — $s_q = 1 + (L'_x / L'_y) \times sin(\phi'_d)$ = **1.365**

$s_\gamma = 1 - 0.3 \times (L'_x / L'_y)$ = **0.739**

$s_c = (s_q \times N_q - 1) / (N_q - 1)$ = **1.403**

Load inclination factors — H = **0.0** kN

$m_y = [2 + (L'_y / L'_x)] / [1 + (L'_y / L'_x)]$ = **1.465**

$m_x = [2 + (L'_x / L'_y)] / [1 + (L'_x / L'_y)]$ = **1.535**

$m = m_x$ = **1.535**

$i_q = [1 - H / (F_{dz} + A' \times c'_d \times cot(\phi'_d))]^m$ = **1.000**

$i_\gamma = [1 - H / (F_{dz} + A' \times c'_d \times cot(\phi'_d))]^{m+1}$ = **1.000**

$i_c = i_q - (1 - i_q) / (N_c \times tan(\phi'_d))$ = **1.000**

Ultimate bearing capacity — $n_f = c'_d \times N_c \times s_c \times i_c + q' \times N_q \times s_q \times i_q + 0.5 \times \gamma_{soil} \times L'_x \times N_\gamma \times s_\gamma \times i_\gamma$ = **368.0** kN/m^2

PASS - Ultimate bearing capacity exceeds design base pressure

UNREINFORCED PAD FOUNDATION FOR MASONRY COLUMN

Foundation design in accordance with EN1992-1-1:2004 + A1:2014 incorporating corrigenda January 2008, November 2010 and January 2014 and the recommended values

Tedds calculation version 3.3.05

Concrete details (Table 3.1 - Strength and deformation characteristics for concrete)

Concrete strength class — C30/37

Characteristic compressive cylinder strength — f_{ck} = **30** N/mm^2

Mean value of axial tensile strength — $f_{ctm} = 0.3 N/mm^2 \times (f_{ck} / 1 N/mm^2)^{2/3}$ = **2.9** N/mm^2

5% fractile of axial tensile strength — $f_{ctk,0.05} = 0.7 \times f_{ctm}$ = **2.0** N/mm^2

Partial factor for concrete (Table 2.1N) γ_C = **1.50**

Tens.strength coeff.for plain concrete (cl.12.3.1(1)) $\alpha_{ct,pl}$ = **0.80**

Des.tens.strength for plain concrete (exp.12.1) $f_{ctd,pl} = \alpha_{ct,pl} \times f_{ctk,0.05} / \gamma_C$ = **1.1** N/mm^2

Strip and pad footings (Section 12.9.3)

Design base pressure f_{dz} = **228.8** kN/m^2

Projection from column face a = **830** mm

Max.projection from column face - (exp.12.13) $a_{max} = 0.85 \times h / \sqrt{[3 \times f_{dz} / f_{ctd,pl}]}$ = **1067** mm

PASS - Projection from the column face doesn't exceed permissible limit for plain concrete

MASONRY COLUMN DESIGN

In accordance with EN1996-1-1:2005 incorporating corrigenda February 2006 and July 2009 and the UK national annex

Tedds calculation version 1.0.07

Design summary

Description	Unit	Capacity	Maximum	Utilisation	Result
Slenderness		27.0	8.0	0.295	PASS
Vertical loading	kN	361.1	196.5	0.544	PASS

Geometry

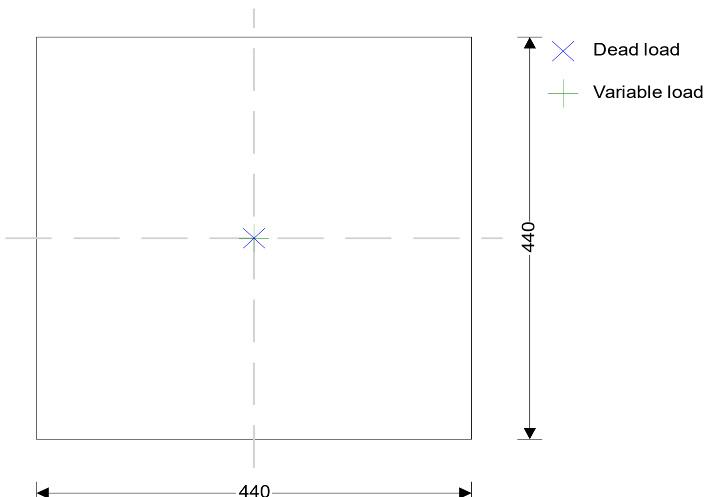
Width of column b = **440** mm

Thickness of column t = **440** mm

Height of column h = **3500** mm

Reduction factor for effective height ρ_2 = **1.0**

Effective height of column (cl 5.5.1.2) $h_{eff} = h \times \rho_2$ = **3500** mm



Plan on column

Loading

Vertical dead load G_k = **120.0** kN

Eccentricity of dead load in x-direction e_{Gb} = **0** mm

Eccentricity of dead load in y-direction e_{Gt} = **0** mm

Vertical live load Q_k = **25.0** kN

Eccentricity of variable load in x-direction e_{Qb} = **0** mm

Eccentricity of variable load in y-direction e_{Qt} = **0** mm

Characteristic wind loading W_k = **0.0** kN/m^2

Vertical wind loading W_v = **0.0** kN

Masonry details

Masonry type **Aggregate concrete brick - Group 1**

Compressive strength of masonry unit f_c = **7.3** N/mm^2

Height of unit h_u = **215** mm

Width of unit w_u = **100** mm

Conditioning factor k = **1.0**

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1 d_{sf} = **1.38**

Mean compressive strength of masonry unit $f_b = f_c \times k \times d_{sf}$ = **10.074** N/mm²

Density of masonry γ = **18** kN/m³

Mortar type **M6 - General purpose mortar**

Compressive strength of masonry mortar f_m = **6** N/mm²

Compressive strength factor - Table NA.4 K = **0.75**

Characteristic compressive strength of masonry - eq 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **6.468** N/mm²

Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6

f_{xk1} = **0.3** N/mm²

Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6

f_{xk2} = **0.9** N/mm²

Partial factors for material strength

Category of manufacturing control **Category II**

Class of execution control **Class 2**

Partial factor for masonry in compressive flexure γ_{Mc} = **3.00**

Slenderness ratio

Slenderness ratio minor axis (cl.5.5.2.1) $\lambda_t = h_{eff} / t$ = **7.95**

Slenderness ratio major axis (cl.5.5.2.1) $\lambda_b = h_{eff} / b$ = **7.95**

Maximum slenderness $\lambda = \max(\lambda_t, \lambda_b)$ = **7.95**

PASS - Slenderness ratio is less than 27

Load combinations derived from Eq.6.10a and Eq.6.10b for lateral and vertical loading (utilisation)

Combination 1 1.35 × perm unfav + 1 × perm fav + 1.5 × 0.7 × variable + 1.5 × 0.5 × wind (0.544)

Combination 2 0.925 × 1.35 × perm unfav + 1 × perm fav + 1.5 × variable + 1.5 × 0.5 × wind (0.540)

Combination 3 0.925 × 1.35 × perm unfav + 1 × perm fav + 1.5 × 0.7 × variable + 1.5 × wind (0.509)

The following output relates to combination 1

Reduction factor for slenderness and eccentricity about the major axis - Section 6.1.2.2

Design bending moment top or bottom of column $M_{idb} = abs(\gamma_{fGv} \times G_k \times e_{Gb} + \gamma_{fQv} \times Q_k \times e_{Qb})$ = **0** kNm

Design vertical load at top or bottom of column $N_{idb} = abs(\gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k)$ = **188.25** kN

Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{eff} / 450$ = **7.8** mm

Conservativley assume moment due to wind load at the top of the column is equal to that at mid height

Design moment due to horizontal load M_{Edb} = **0** kNm

Eccentricity due to horizontal load e_{hb} = **0.0** mm

Eccentricity at top or bottom of column - eq.6.5 $e_{ib} = \max(M_{idb} / N_{idb} + e_{hb} + e_{init}, 0.05 \times b)$ = **22.0** mm

Reduction factor top or bottom of column - eq.6.4 $\Phi_{ib} = \max(1 - 2 \times e_{ib} / b, 0)$ = **0.9**

Ratio of top and middle mnts due to eccentricity α_{mdb} = **1.0**

Design bending moment at middle of column $M_{mdb} = \alpha_{mdb} \times abs(\gamma_{fGv} \times G_k \times e_{Gb} + \gamma_{fQv} \times Q_k \times e_{Qb})$ = **0** kNm

Design vertical load at middle of column $N_{mdb} = \gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k + \gamma_{fGv} \times t \times b \times \gamma \times h / 2$ = **196.483** kN

Eccentricity due to horizontal load e_{hmb} = **0.0** mm

Eccentricity middle of column due to loads - eq.6.7 $e_{mb} = M_{mdb} / N_{mdb} + e_{hmb} + e_{init}$ = **7.8** mm

Eccentricity at middle of column due to creep e_{kb} = **0.0** mm

Eccentricity at middle of column - eq.6.6 $e_{mkb} = \max(e_{mb} + e_{kb}, 0.05 \times b)$ = **22.0** mm

From eq.G.2 $A_{1b} = 1 - 2 \times e_{mkb} / b$ = **0.9**

Short term secant modulus of elasticity factor K_E = **1000**

Modulus of elasticity - cl.3.7.2 $E = K_E \times f_k$ = **6468** N/mm²

Slenderness - eq.G.4 $\lambda_b = (h_{eff} / b) \times \sqrt{(f_k / E)}$ = **0.252**

From eq.G.3 $u_b = (\lambda_b - 0.063) / (0.73 - 1.17 \times e_{mkb} / b)$ = **0.281**

Reduction factor at middle of column - eq.G.1 $\Phi_{mb} = \max(A_{1b} \times e_e^{-(u_b \times u_b)/2}, 0)$ = **0.865**

Reduction factor for slenderness and eccentricity $\Phi_b = \min(\Phi_{ib}, \Phi_{mb})$ = **0.865**

Reduction factor for slenderness and eccentricity about the minor axis - Section 6.1.2.2

Design bending moment top or bottom of column $M_{idt} = abs(\gamma_{fGv} \times G_k \times e_{Gt} + \gamma_{fQv} \times Q_k \times e_{Qt})$ = **0** kNm

Design vertical load at top or bottom of column $N_{idt} = abs(\gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k)$ = **188.25** kN

Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{eff} / 450$ = **7.8** mm

Conservativley assume moment due to wind load at the top of the column is equal to that at mid height

Design moment due to horizontal load M_{Edt} = **0** kNm

Eccentricity due to horizontal load e_{ht} = **0.0** mm

Eccentricity at top or bottom of column - eq.6.5 $e_{it} = \max(M_{idt} / N_{idt} + e_{ht} + e_{init}, 0.05 \times t)$ = **22.0** mm

Reduction factor top or bottom of column - eq.6.4 $\Phi_{it} = \max(1 - 2 \times e_{it} / t, 0)$ = **0.9**

Ratio of top and middle mnts due to eccentricity α_{mdt} = **1.0**

Design bending moment at middle of column $M_{mdt} = \alpha_{mdt} \times abs(\gamma_{fGv} \times G_k \times e_{Gt} + \gamma_{fQv} \times Q_k \times e_{Qt})$ = **0** kNm

Design vertical load at middle of column $N_{mdt} = \gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k + \gamma_{fGv} \times t \times b \times \gamma \times h / 2$ = **196.483** kN

Eccentricity due to horizontal load e_{hmt} = **0.0** mm

Eccentricity middle of column due to loads - eq.6.7 $e_{mt} = M_{mdt} / N_{mdt} + e_{hmt} + e_{init}$ = **7.8** mm

Eccentricity at middle of column due to creep e_{kt} = **0.0** mm

Eccentricity at middle of column - eq.6.6 $e_{mkt} = \max(e_{mt} + e_{kt}, 0.05 \times t)$ = **22.0** mm

From eq.G.2 $A_{1t} = 1 - 2 \times e_{mkt} / t$ = **0.9**

Short term secant modulus of elasticity factor K_E = **1000**

Modulus of elasticity - cl.3.7.2 $E = K_E \times f_k$ = **6468** N/mm^2

Slenderness - eq.G.4 $\lambda_t = (h_{eff} / t) \times \sqrt{(f_k / E)}$ = **0.252**

From eq.G.3 $u_t = (\lambda_t - 0.063) / (0.73 - 1.17 \times e_{mkt} / t)$ = **0.281**

Reduction factor at middle of column - eq.G.1 $\Phi_{mt} = \max(A_{1t} \times e_e^{-(u_t \times u_t)/2}, 0)$ = **0.865**

Reduction factor for slenderness and eccentricity $\Phi_t = \min(\Phi_{it}, \Phi_{mt})$ = **0.865**

Columns subjected to mainly vertical loading - Section 6.1.2

Design value of the vertical load $N_{Ed} = \max(N_{idb}, N_{mdb}, N_{idt}, N_{mdt})$ = **196.483** kN

Design compressive strength of masonry $f_d = f_k / \gamma_{Mc}$ = **2.156** N/mm^2

Vertical resistance of column - eq.6.2 $N_{Rd} = \min(\Phi_t, \Phi_b) \times t \times b \times f_d$ = **361.123** kN

Utilisation N_{Ed} / N_{Rd} = 0.544

PASS - Design vertical resistance exceeds applied design vertical load

Padstone calculator The Old Coach House 20 June 2023

Masonry Input

Structural calculations for padstones

Beam End Reaction	122	kN (factored)
Characteristic strength of masonry	5.4	N/mm^2
Width of beam end bearing	330	mm
Length of beam end bearing	200	mm

γm = 3.0

Bearing Factor = 1.25

Masonry Results

Maximum Bearing Stress:	2.25	N/mm^2
Actual Bearing Stress:	1.85	N/mm^2

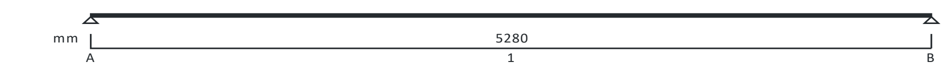
Padstone Not Required

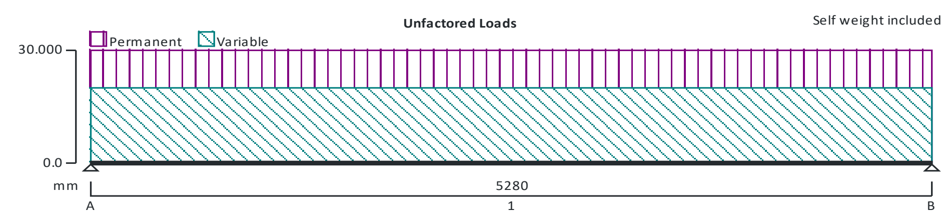
Use high compressive strength concrete blocks for columns (10KN or greater) and have steel beam bearing of 200mm or greater. No high Newton padstone will be required as per calculations.

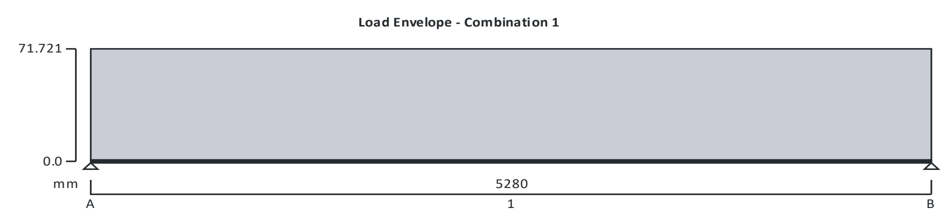
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

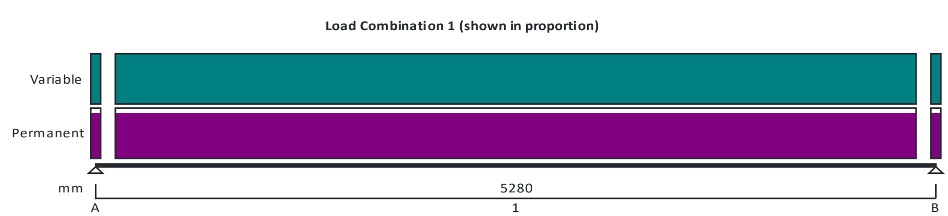
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

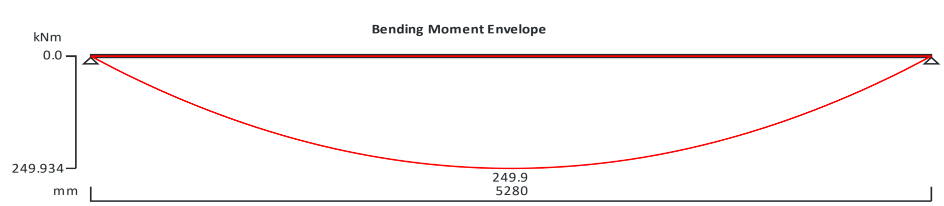
TEDDS calculation version 3.0.14



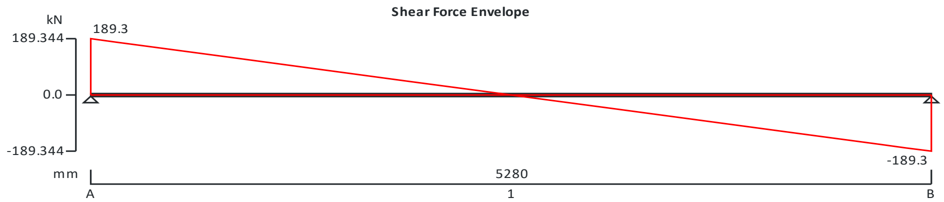








PlanningApplications.com Summer House, Upper Court Road Woldingham SURREY CR3 7BF support@planningapplications.com 07922 148 701	Project BEAM 1 (2 No. 305x165x46kgUB) bolted togther min 1.3m				Job no. 241	
	Calcs for The Old Coach House - Dylan & Poppy				Start page no./Revision 2	
	Calcs by SB	Calcs date 19/06/2023	Checked by DB	Checked date 19/06/2023	Approved by SB	Approved date 19/06/2023



Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 30 kN/m
	Variable full UDL 20 kN/m

Load combinations

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

Analysis results

Maximum moment	M_{max} = **249.9** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **187.5** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **249.9** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum moment span 1 segment 3	$M_{s1\_seg3\_max}$ = **249.9** kNm	$M_{s1\_seg3\_min}$ = **0** kNm
Maximum moment span 1 segment 4	$M_{s1\_seg4\_max}$ = **187.5** kNm	$M_{s1\_seg4\_min}$ = **0** kNm
Maximum shear	V_{max} = **189.3** kN	V_{min} = **-189.3** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **189.3** kN	$V_{s1\_seg1\_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **94.7** kN	$V_{s1\_seg2\_min}$ = **0** kN
Maximum shear span 1 segment 3	$V_{s1\_seg3\_max}$ = **0** kN	$V_{s1\_seg3\_min}$ = **-94.7** kN
Maximum shear span 1 segment 4	$V_{s1\_seg4\_max}$ = **0** kN	$V_{s1\_seg4\_min}$ = **-189.3** kN
Deflection segment 5	δ_{max} = **4.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **189.3** kN	R_{A_min} = **189.3** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **81.6** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **52.8** kN	
Maximum reaction at support B	R_{B_max} = **189.3** kN	R_{B_min} = **189.3** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **81.6** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **52.8** kN	

Section details

Section type	**2 x UB 305x165x46 (BS4-1)**
Steel grade	**S275**

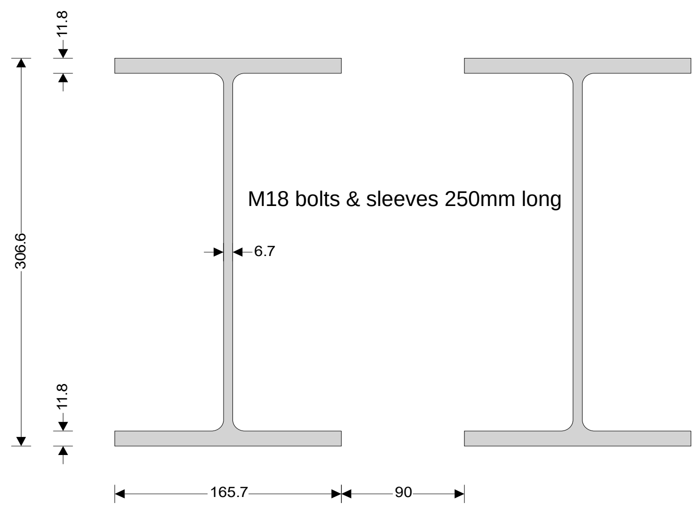
EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element $t = max(t_f, t_w) = \mathbf{11.8}$ mm

Nominal yield strength $f_y = \mathbf{275}$ N/mm^2

Nominal ultimate tensile strength $f_u = \mathbf{410}$ N/mm^2

Modulus of elasticity $E = \mathbf{210000}$ N/mm^2



Both beams bolted together every 400mm with M18 bolts with hollow steel tube as spacer sleeves 250mm long. Total beam width 421.4mm. Sleeves can be shortened if preferred.

Partial factors - Section 6.1

Resistance of cross-sections $\gamma_{M0} = \mathbf{1.00}$

Resistance of members to instability $\gamma_{M1} = \mathbf{1.00}$

Resistance of tensile members to fracture $\gamma_{M2} = \mathbf{1.10}$

Lateral restraint

Span 1 has lateral restraint at supports plus 1320 mm, 2640 mm and 3960 mm

Effective length factors

Effective length factor in major axis $K_y = \mathbf{1.000}$

Effective length factor in minor axis $K_z = \mathbf{1.000}$

Effective length factor for torsion $K_{LT.A} = \mathbf{1.000}$

$K_{LT.B} = \mathbf{1.000}$

Classification of cross sections - Section 5.5

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

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

Width of section $c = d = \mathbf{265.2}$ mm

$\alpha = \min([h / 2 + N_{Ed} / (2 \times N \times t_w \times f_y) - (t_f + r)] / c, 1) = \mathbf{0.653}$

$c / t_w = 42.8 \times \varepsilon <= 396 \times \varepsilon / (13 \times \alpha - 1)$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2 = \mathbf{70.6}$ mm

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

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f = \mathbf{283}$ mm

Shear area factor	η = **1.000**
	$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{Ed} = max(abs(V_{max}), abs(V_{min}))$ = **189.3** kN
Shear area - cl 6.2.6(3)	$A_v = max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **2253** mm^2
Design shear resistance - cl 6.2.6(2)	$V_{c,Rd} = V_{pl,Rd} = N \times A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **715.5** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 1 segment 2 major (y-y) axis - Section 6.2.5

Design bending moment	$M_{Ed} = max(abs(M_{s1\_seg2\_max}), abs(M_{s1\_seg2\_min}))$ = **249.9** kNm
Design bending resistance moment - eq 6.13	$M_{c,Rd} = M_{pl,Rd} = N \times W_{pl.y} \times f_y / \gamma_{M0}$ = **396** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6	k_c = **0.969**
	$C_1 = 1 / k_c^2$ = **1.065**
Curvature factor	$g = \sqrt{[1 - (I_z / I_y)]}$ = **0.954**
Poissons ratio	ν = **0.3**
Shear modulus	$G = E / [2 \times (1 + \nu)]$ = **80769** N/mm^2
Unrestrained length	$L = 1.0 \times L_{s1\_seg2}$ = **1320** mm
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **1819.9** kNm
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **0.33**
Limiting slenderness ratio	$\bar{\lambda}_{LT,0}$ = **0.4**

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5	b
Imperfection factor - Table 6.3	α_{LT} = **0.34**
Correction factor for rolled sections	β = **0.75**
LTB reduction determination factor	$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **0.529**
LTB reduction factor - eq 6.57	$\chi_{LT} = min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **1.000**
Modification factor	$f = min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **0.991**
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = min(\chi_{LT} / f, 1)$ = **1.000**
Design buckling resistance moment - eq 6.55	$M_{b,Rd} = \chi_{LT,mod} \times N \times W_{pl.y} \times f_y / \gamma_{M1}$ = **396** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check compression - Section 6.2.4

Design compression force	N_{Ed} = **300** kN
Design resistance of section - eq 6.10	$N_{c,Rd} = N_{pl,Rd} = N \times A \times f_y / \gamma_{M0}$ = **3231** kN

Slenderness ratio for major (y-y) axis buckling

Critical buckling length	$L_{cr,y} = L_{s1} \times K_y$ = **5280** mm
Critical buckling force	$N_{cr,y} = \pi^2 \times E_{SEC3} \times I_y / L_{cr,y}^2$ = **7359.2** kN
Slenderness ratio for buckling - eq 6.50	$\bar{\lambda}_y = \sqrt{[A \times f_y / N_{cr,y}]}$ = **0.469**

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2	a
Imperfection factor - Table 6.1	α_y = **0.21**
Buckling reduction determination factor	$\phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2]$ = **0.638**
Buckling reduction factor - eq 6.49	$\chi_y = min(1 / [\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}], 1)$ = **0.934**

Design buckling resistance - eq 6.47 $N_{b,y,Rd} = \chi_y \times N \times A \times f_y / \gamma_{M1}$ = **3017** kN

PASS - Design buckling resistance exceeds design compression force

Slenderness ratio for minor (z-z) axis buckling

Critical buckling length $L_{cr,z} = L_{s1\_seg2} \times K_z$ = **1320** mm

Critical buckling force $N_{cr,z} = \pi^2 \times E_{SEC3} \times I_z / L_{cr,z}^2$ = **10654.1** kN

Slenderness ratio for buckling - eq 6.50 $\bar{\lambda}_z = \sqrt{[A \times f_y / N_{cr,z}]}$ = **0.389**

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 α_z = **0.34**

Buckling reduction determination factor $\phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$ = **0.608**

Buckling reduction factor - eq 6.49 $\chi_z = \min(1 / [\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}], 1)$ = **0.930**

Design buckling resistance - eq 6.47 $N_{b,z,Rd} = \chi_z \times N \times A \times f_y / \gamma_{M1}$ = **3005.7** kN

PASS - Design buckling resistance exceeds design compression force

Check torsional and torsional-flexural buckling - Section 6.3.1.4

Torsional buckling length factor K_T = **1.00**

Torsional buckling length $L_{cr,T} = \max(L_{s1}, L_{s1\_seg2}) \times K_T$ = **5280** mm

Distance from shear centre to centroid in y axis y_0 = **0.0** mm

Distance from shear centre to centroid in z axis z_0 = **0.0** mm

Radius of gyration $i_0 = \sqrt{[i_y^2 + i_z^2]}$ = **135.6** mm

Elastic critical torsional buckling force $N_{cr,T} = 1 / i_0^2 \times [G \times I_t + \pi^2 \times E_{SEC3} \times I_w / L_{cr,T}^2]$ = **1763.1** kN

Torsion factor $\beta_T = 1 - (y_0 / i_0)^2$ = **1.000**

Elastic critical torsional-flexural buckling force

$N_{cr,TF} = N_{cr,y} / (2 \times \beta_T) \times [1 + N_{cr,T} / N_{cr,y} - \sqrt{[(1 - N_{cr,T} / N_{cr,y})^2 + 4 \times (y_0 / i_0)^2 \times N_{cr,T} / N_{cr,y}]}]$ = **1763.1** kN

Elastic critical buckling force $N_{cr} = \min(N_{cr,T}, N_{cr,TF})$ = **1763.1** kN

Slenderness ratio for torsional buckling - eq 6.52 $\bar{\lambda}_T = \sqrt{[A \times f_y / N_{cr}]}$ = **0.957**

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2 b

Imperfection factor - Table 6.1 α_T = **0.34**

Buckling reduction determination factor $\phi_T = 0.5 \times [1 + \alpha_T \times (\bar{\lambda}_T - 0.2) + \bar{\lambda}_T^2]$ = **1.087**

Buckling reduction factor - eq 6.49 $\chi_T = \min(1 / [\phi_T + \sqrt{(\phi_T^2 - \bar{\lambda}_T^2)}], 1)$ = **0.624**

Design buckling resistance - eq 6.47 $N_{b,T,Rd} = \chi_T \times N \times A \times f_y / \gamma_{M1}$ = **2017.3** kN

PASS - Design buckling resistance exceeds design compression force

Combined bending and axial force - Section 6.2.9

Bending and axial force check - eq 6.33 & 6.34 $N_{Ed} <= \min(0.25 \times N_{pl,Rd}, 0.5 \times N \times h_w \times t_w \times f_y / \gamma_{M0})$

No allowance on the plastic moment need to be accounted for due to the effect of axial force

Check combined bending and compression - Section 6.3.3

Equivalent uniform moment factors - Table B.3 M_{hy} = **0** kNm

M_{sy} = **250** kNm

ψ_y = **1.000**

$\alpha_{hy} = M_{hy} / M_{sy}$ = **0.000**

$C_{my} = 0.95 + 0.05 \times \alpha_{hy}$ = **0.950**

M_{hz} = **0** kNm

M_{sz} = **0** kNm

ψ_z = **1.000**

$C_{mz} = 0.6 + 0.4 \times \psi_z = \mathbf{1.000}$

$M_{hLT} = \mathbf{250}$ kNm

$M_{sLT} = \mathbf{234}$ kNm

$\psi_{LT} = \mathbf{0.750}$

$\alpha_{sLT} = M_{sLT} / M_{hLT} = \mathbf{0.937}$

$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{sLT}, 0.4) = \mathbf{0.950}$

Interaction factors k_{ij} for members susceptible to torsional deformations - Table B.2

Characteristic moment resistance $M_{Rk} = N \times W_{pl.y} \times f_y = \mathbf{396}$ kNm

Characteristic resistance to normal force $N_{Rk} = N \times A \times f_y = \mathbf{3231}$ kN

Interaction factors $k_{yy} = C_{my} \times [1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})] = \mathbf{0.975}$

$k_{zy} = \min(0.6 + \bar{\lambda}_z, 1 - 0.1 \times \bar{\lambda}_z \times N_{Ed} / ((C_{mLT} - 0.25) \times \chi_z \times N_{Rk} / \gamma_{M1})) =$ **0.989**

Interaction formulae - eq 6.61 & eq 6.62 $N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1}) = \mathbf{0.715}$

$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1}) = \mathbf{0.724}$

PASS - Combined bending and compression checks are satisfied

Check vertical deflection - Section 7.2.1

Consider deflection due to variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 = \mathbf{14.7}$ mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{4.868}$ mm

PASS - Maximum deflection does not exceed deflection limit

Loading details

Self weight

Permanent action SW = w × 9.81 / 1000 = 0.452 kN/m

Load 1: UDL - Sloping roof, 0° to 30°

Permanent action G1 = 1.15 kN/m² × 3.21 m = 3.69 kN/m

Variable action Q1 = 0.75 kN/m² × 3.21 m = 2.41 kN/m

Load 2: UDL - Flat roof, with no permanent access

Permanent action G2 = 1 kN/m² × 1.25 m = 1.25 kN/m

Variable action Q2 = 0.75 kN/m² × 1.25 m = 0.938 kN/m

Load 3: UDL - 460mm thick stone wall x 2500mm tall

Permanent action G3 = 10 kN/m² × 2.5 m = 25 kN/m

Variable action Q3 = 0 kN/m² × 2.5 m = 0 kN/m

Reactions

	Permanent (unfactored)	Variable (unfactored)	Total (unfactored)	Total (factored)
Left reaction	80.2 kN	8.83 kN	89.1 kN	122 kN
Right reaction	80.2 kN	8.83 kN	89.1 kN	122 kN