CIVIL & STRUCTURAL CONSULTING ENGINEERS

The Pavilion

Structural Calculations

For

Winterbourne Parish Council

Date:

12/02/2025

Project Reference: M2-3168

Document Reference: SE-01_A1

Web: m2structural.co.uk
Phone: 01225 285073

Document Issue History and Status

	Prepared By		Checked By		Approved By	
Revision	**Name**	**Date**	**Name**	**Date**	**Name**	**Date**
A1	**Matthieu Crosnier**	03.02.25	**Max Day**	12.02.25	**Matthieu Crosnier**	12.02.25

General Notes:

Calculations to be checked by Building Control Authority before work commences. Client to ensure all of contractors' works on site comply with and meet Approval of the relevant British Standards and the Local Authority including Building Control and Planning Departments. All temporary works to be the responsibility of the contractor and should be in accordance with BS 5975 to ensure temporary stability during the course of the works.

Dimensions: Note that all dimensions shown on the calculations are indicative and have been used for calculation purposes only. All dimensions should be checked prior to start of the works on site. It is the responsibility of the client to notify the Engineer of any discrepancies. The same applies to the alignment of walls and general layouts. All existing foundations and lintels to be exposed to verify suitability and to be checked for adequacy and/or replaced or surrounded in 150mm concrete cover if necessary. Prior to commencement a trial hole and /or soil report/investigation and an inspection of any trees in the areas may be required.

Introduction:

The following design calculations are for the single storey extension at the sport Pavilion in Winterbourne, Bristol.

Design Codes - Eurocodes:

BS EN 1990:	Eurocode 0:	“Basis of Structural Design”
BS EN 1991:	Eurocode 1:	“Actions on Structures”
BS EN 1992:	Eurocode 2:	“Design of Concrete Structures”
BS EN 1993:	Eurocode 3:	“Design of Steel Structures”
BS EN 1994:	Eurocode 4:	“Design of Composite Steel and Concrete Structures”
BS EN 1995:	Eurocode 5:	“Design of Timber Structures”
BS EN 1996:	Eurocode 6:	“Design of Masonry Structures”
BS EN 1997:	Eurocode 7:	“Geotechnical Design”
BS EN 1998:	Eurocode 8:	“Design of Structures for Earthquake Resistance”
BS EN 1999:	Eurocode 9:	“Design of Aluminium Structures”

Loadings

Roof Loading (Pitched Roof)

Roof slope; A = **5°**

Dead Load

Metal sheeting;	$Roof_{D1}$	=	**0.15** kN/m^2
Felt & Boarding;	$Roof_{D2}$	=	**0.05** kN/m^2
Rafters;	$Roof_{D3}$	=	**0.12** kN/m^2
Insulation;	$Roof_{D4}$	=	**0.05** kN/m^2
Services;	$Roof_{D5}$	=	**0.05** kN/m^2
Plasterboard & Skim;	$Roof_{D6}$	=	**0.15** kN/m^2
Dead Load on slope;	$Roof_{DL\_sroof} = sum(Roof_{D1},Roof_{D2},Roof_{D3},Roof_{D4},Roof_{D5},Roof_{D6})$	=	**0.570** kN/m^2
Total Dead Load (on plan);	$Roof_{DL} = Roof_{DL\_sroof} / \cos(A)$	=	**0.572** kN/m^2

Imposed Load

Roof Imposed Load (on plan);	$Roof_{IL1}$	=	**0.60** kN/m^2

Cavity Wall Loading

Dead Load

Masonry (Outer Leaf);	CW_{D1}	=	**2.20** kN/m^2
Insulation;	CW_{D2}	=	**0.05** kN/m^2
Masonry (Inner Leaf);	CW_{D3}	=	**1.80** kN/m^2
Plasterboard & Skim;	CW_{D4}	=	**0.15** kN/m^2
Total Dead Load;	$CW_{DL} = sum(CW_{D1},CW_{D2},CW_{D3},CW_{D4})$	=	**4.20** kN/m^2

Internal Blockwork Wall Loading

Dead Load

Masonry;	IW_{D1}	=	**1.80** kN/m^2
Plasterboard & Skim (both sides);	IW_{D2}	=	**0.30** kN/m^2
Total Dead Load;	$IW_{DL} = sum(IW_{D1},IW_{D2})$	=	**2.10** kN/m^2

Stud Wall Loading

Dead Load

Stud;	SW_{D1}	=	**0.20** kN/m^2
Plasterboard & Skim (both sides);	SW_{D2}	=	**0.30** kN/m^2
Total Dead Load;	$SW_{DL} = sum(SW_{D1},SW_{D2})$	=	**0.50** kN/m^2

Beam B1 Design:

Loadings

UDL from Pitched Roof

Width of roof being carried; x_1 = **3.7** m

Dead Load; $DL_1 = 0.572\ kN/m^2 \times x_1$ = **2.116** kN/m

Imposed Load; $IL_1 = 0.60\ kN/m^2 \times x_1$ = **2.220** kN/m

B1 STEEL MASONRY SUPPORT (EN1993)

STEEL MASONRY SUPPORT

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

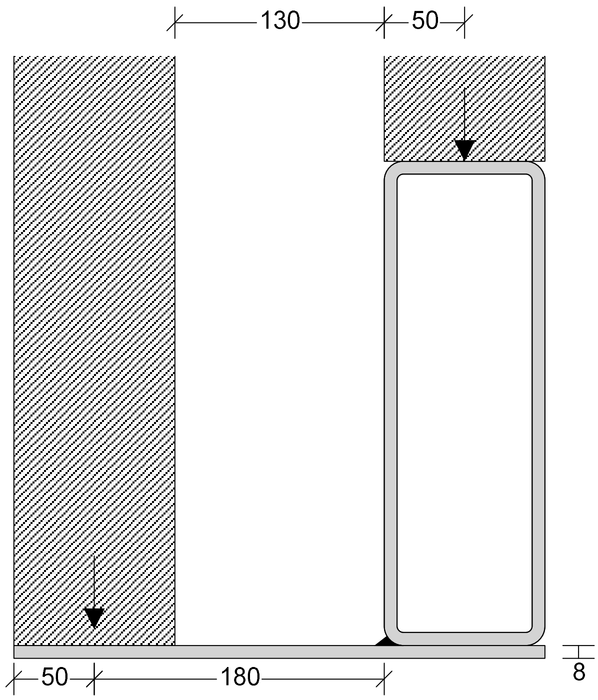
Tedds calculation version 1.0.05

Design summary

Overall design status; PASS

Overall design utilisation; 0.763

Description	Unit	Allowable	Applied	Utilisation	Result
Heel moment	kNm/m	2.933	0.680	0.232	PASS
Deflection	mm	1.8	0.9	0.486	PASS
Weld capacity	kN/m	803.4	296.7	0.369	PASS
Shear force (major axis)	kN	695.9	38.1	0.055	PASS
Bending (major axis)	kNm	150.1	49.5	0.330	PASS
Bending (minor axis)	kNm	67.5	0.1	0.001	PASS
Torsion resistance	kNm	61.4	2.3	0.038	PASS
Plastic interaction				0.159	PASS
Torsion beam rotation	deg	3.00	0.05	0.017	PASS
Torsion beam deflection	mm	10.0	7.6	0.763	PASS



Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = **1**

Resist. of members to instability; γ_{M1} = **1**

Resistance of joints; γ_{M2} = **1.25**

Partial factor for permanent action; γ_G = **1.35**

Partial factor for variable action; γ_Q = **1.50**

Steel beam section details

Torsion beam section type; RHS 300x100x8.0

Nominal yield strength $f_y = f_{y,tb}$ = **275** N/mm^2

Nominal ultimate tensile strength $f_u = f_{u,tb}$ = **410** N/mm^2

Masonry support section details

Section type; Plate 330x8(230)

Steel grade; User defined

Nominal thickness; $t_{nom,sb} = t_{plate}$ = **8** mm

Nominal yield strength; $f_{y,sb}$ = **275** N/mm^2

Nominal ultimate tensile strength; $f_{u,sb}$ = **410** N/mm^2

Modulus of elasticity; E_{sb} = **210000** N/mm^2

Total length of plate; l_{plate} = **330** mm

Length of plate beyond outer edge of torsion beam; l_h = **230** mm

Supported materials detail

Density of masonry on torsion beam; $\rho_{m,tb}$ = **20.0** kN/m^3

Width of masonry on torsion beam; $b_{m,tb}$ = **100** mm

Height of masonry on torsion beam; $h_{m,tb}$ = **1400** mm

Eccentricity of torsion beam masonry; $e_{load,tb}$ = **50** mm

Eccentricity of torsion beam material; e_{tb} = **0** mm

Add perm. force torsion beam (not masonry); $G_{k,add,tb}$ = **2.1** kN/m

Add var. force torsion beam (not masonry); $Q_{k,add,tb}$ = **2.2** kN/m

Density of masonry on support beam; $\rho_{m,sb}$ = **20.0** kN/m^3

Width of masonry on support beam; $b_{m,sb}$ = **100** mm

Height of masonry on support beam; $h_{m,sb}$ = **1400** mm

Eccentricity of support beam masonry; $e_{load,sb}$ = **180** mm

Geometry

Cavity width; b_{cavity} = **130** mm

Supported width of masonry; $d_m = l_h + t_{shim} + e_{tb} - b_{cavity}$ = **100** mm

Biaxial stress effects in the plate (SCI-P-110)

Maximum overall bending moment; $M_{y,Ed}$ = **49.5** kNm

Dist to NA combined section (CoG torsion beam); $z_{na,all} = (h_{tb} + t_{plate}) \times A_{pl} / (2 \times (A_{tb} + A_{pl}))$ = **47** mm

Second moment of area of combined section; $I_{y,all} = (I_{ytb} + A_{tb} \times z_{na,all}^2) + A_{pl} \times (h_{tb} / 2 + t_{plate} / 2 - z_{na,all})^2$ = **10670** cm^4

Elastic section modulus of combined section; $Z_{y,all} = I_{y,all} / (h_{tb} / 2 + t_{plate} - z_{na,all})$ = **958.21** cm^3

Section modulus of plate; $Z_{y,plate} = 1m \times t_{plate}^2 / (6 \times 1m)$ = **10.67** cm^3/m

Force of masonry on support plate; $F_1 = (b_{m,sb} \times h_{m,sb} \times \rho_{m,sb} + G_{k,add,sb}) \times \gamma_G + Q_{k,add,sb} \times \gamma_Q$ = **3.8** kN/m

Bending at heel; $M_{y,Ed,plate} = F_1 \times e_{load,sb}$ = **0.7** kNm/m

Moment capacity of plate; $M_{y,Rd,plate} = Z_{y,plate} \times f_{y,sb} / \gamma_{M0}$ = **2.9** kNm/m

PASS - Moment capacity of plate exceeds applied moment

Longitudinal stress due to overall bending; $\sigma_1 = M_{y,Ed} / Z_{y,all}$ = **51.7** N/mm^2

Constant relating to Von Mises curve; $c_{fp} = (4 \times f_{y,sb}^2 - 3 \times \sigma_1^2)^{0.5}$ = **542.7** N/mm^2

Transverse bending stress ratio limit; $\alpha_{ts} = (c_{fp}^2 - \sigma_1^2) / (2 \times c_{fp} \times f_{y,sb})$ = **0.978**

Transverse bending stress ratio; $\alpha_{ls} = M_{y,Ed,plate} / M_{y,Rd,plate}$ = **0.232**

PASS - Transverse bending stress ratio less than allowable limit

Deflection of plate

Unfactored force on support angle; $F_{1ser} = (b_{m,sb} \times h_{m,sb} \times \rho_{m,sb} + G_{k,add,sb}) + Q_{k,add,sb}$ = **2.8** kN/m

Distance from weld to load position; $a_m = e_{load,sb}$ = **180** mm

Length of load resultant to edge of plate; $b_m = l_h - e_{load,sb}$ = **50** mm

Dist from weld to load position as ratio of length; $a_l = a_m / (a_m + b_m)$ = **0.783**

Effective second moment of area; $I_{eff\_def} = t_{plate}^3 / 12$ = **42667** mm^4/m

Deflection at end of plate; $\delta = (a_l^2 \times (3 - a_l) / 6) \times (F_{1ser} \times (a_m + b_m)^3) / (E_{sb} \times I_{eff\_def})$ = **0.86** mm

Deflection limit; $\delta_{lim} = \min((1 + d_m / b_{cavity}) \times 1mm, 2mm)$ = **1.77** mm

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Shear force at weld position; $F_A = F_1 \times \max((1 + (3 \times e_{load,sb}) / (2 \times b_{tb} / 2)), 1.4)$ = **24.2** kN/m

Maximum possible force in plate; $F_p = (l_h + \min(b_{tb}, l_{plate} - l_h)) \times t_{plate} \times f_{y,sb}$ = **726.0** kN

Longitudinal shear between beam and plate; $F_l = 2 \times F_p / L =$ **279.2** kN/m

Horizontal shear between beam and plate; $F_h = F_1 \times e_{load,sb} / (s_{weld} / 2 + t_{plate} / 2) =$ **97.2** kN/m

Resultant weld force; $F_{w,Ed} = (F_A^2 + F_l^2 + F_h^2)^{0.5} =$ **296.7** kN/m

Leg length of weld; $s_{weld} =$ **6.00** mm

Throat thickness of weld; $a_{weld} = 1 / \sqrt{(2)} \times s_{weld} =$ **4.24** mm

Length of weld per metre run; $l_{weld} =$ **1000** mm/m

Ultimate tensile strength used for weld; $f_{u,weld} = \min(f_{u,sb}, f_{u,tb}) =$ **410.0** N/mm^2

Correlation factor (table 4.1); $\beta_w =$ **1.00**

Design shear strength; $f_{vw,d} = f_{u,weld} / (\sqrt{(3)} \times \beta_w \times \gamma_{M2}) =$ **189.4** N/mm^2

Design resistance of weld; $F_{w,Rd} = f_{vw,d} \times a_{weld} =$ **803.4** kN/m

PASS - weld capacity exceeds applied force

Eccentricities

Distance to shear centre of torsion beam; $e_{0,tb} =$ **0** mm

Eccentricity of support beam masonry; $e_{m,sb} = e_{load,sb} + b_{tb} / 2 =$ **230** mm

Eccentricity of torsion beam masonry; $e_{m,tb} = b_{tb} / 2 - e_{load,tb} =$ **0** mm

Eccentricity of support beam; $e_{b,sb} = c_{zsb} + b_{tb} / 2 =$ **115** mm

Eccentricity of torsion beam; $e_{b,tb} =$ **0** mm

Torsional loading ULS

Loading of support beam masonry; $w_{sb} = (h_{m,sb} \times b_{m,sb} \times \rho_{m,sb}) \times \gamma_G =$ **3.78** kN/m

Loading of torsion beam masonry; $w_{tb} = (h_{m,tb} \times b_{m,tb} \times \rho_{m,tb} + G_{k,add,tb}) \times \gamma_G + Q_{k,add,tb} \times \gamma_Q =$ **9.97** kN/m

Self weight of support beam; $w_{sw,sb} = A_{pl} \times \rho_{SEC3} \times g_{acc} \times \gamma_G =$ **0.27** kN/m

Self weight of torsion beam; $w_{sw,tb} = A_{tb} \times \rho_{SEC3} \times g_{acc} \times \gamma_G =$ **0.63** kN/m

Torsional loading SLS

Loading of support beam masonry; $w_{sb,ser} = h_{m,sb} \times b_{m,sb} \times \rho_{m,sb} =$ **2.80** kN/m

Loading of torsion beam masonry; $w_{tb,ser} = h_{m,tb} \times b_{m,tb} \times \rho_{m,tb} + G_{k,add,tb} + Q_{k,add,tb} =$ **7.14** kN/m

Self weight of support beam; $w_{sw,sb,ser} = A_{pl} \times \rho_{SEC3} \times g_{acc} =$ **0.20** kN/m

Self weight of torsion beam; $w_{sw,tb,ser} = A_{tb} \times \rho_{SEC3} \times g_{acc} =$ **0.47** kN/m

Torsional effects

Applied torque (ULS); $T_{d,w} = \text{abs}(w_{sb} \times e_{m,sb} + w_{tb} \times e_{m,tb} + w_{sw,sb} \times e_{b,sb} + w_{sw,tb} \times e_{b,tb}) =$ **0.90** kNm/m

Total torque (ULS); $T_d = T_{d,w} \times L =$ **4.68** kNm

Applied torque (SLS); $T_{d,w,ser} = \text{abs}(w_{sb,ser} \times e_{m,sb} + w_{tb,ser} \times e_{m,tb} + w_{sw,sb,ser} \times e_{b,sb} + w_{sw,tb,ser} \times e_{b,tb}) =$ **0.67** kNm/m

Total torque (SLS); $T_{d,ser} = T_{d,w,ser} \times L =$ **3.47** kNm

STEEL BEAM TORSION DESIGN (EN1993)

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

Tedds calculation version 1.0.05

Partial factors - Section 6.1

Resistance of cross-sections; $\gamma_{M0} =$ **1**

Resistance of members to instability; $\gamma_{M1} =$ **1**

Section details

Section type; RHS 300x100x8.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))

Steel grade; User defined

Nominal thickness of element; $t_{nom} = t = \mathbf{8}$ 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

Shear centre

Distance between flange shear centres $h_s = h - t = \mathbf{292.0}$ mm

Shear centre (above bottom flange centroid); $e_{s,bf} = h_s / 2 = \mathbf{146.0}$ mm

Torsional section modulus

Perimeter length; $p = 2 \times ((h - t) + (b - t)) - 2 \times 1.25 \times t \times (4 - \pi) = \mathbf{751}$ mm

Area enclosed by mean perimeter; $A_p = (h - t) \times (b - t) - (1.25 \times t)^2 \times (4 - \pi) = \mathbf{26778}$ mm^2

Torsional section modulus; $W_t = I_t / (t + 2 \times A_p / p) = \mathbf{387}$ cm^3

Analysis results

Design bending moment - major axis; $M_{y,Ed} = \mathbf{49.5}$ kNm

Design shear force - major axis; $V_{y,Ed} = \mathbf{38.1}$ kN

Classification

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

Width of section; $c = h - 3 \times t = \mathbf{276}$ mm

$c / t = 34.5 = 37.3 \times \varepsilon <= 72 \times \varepsilon$; Class 1

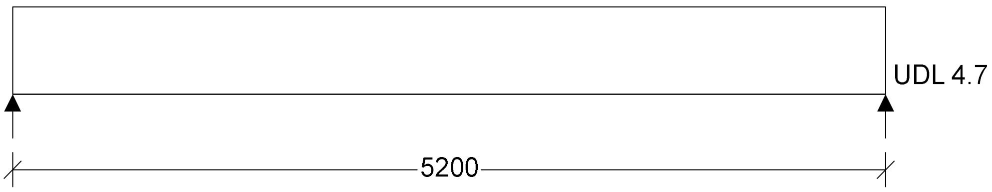
Internal compression parts subject to compression - Table 5.2 (sheet 1 of 3)

Width of section; $c = b - 3 \times t = \mathbf{76}$ mm

$c / t = 9.5 = 10.3 \times \varepsilon <= 33 \times \varepsilon$; Class 1

Section is class 1

Torsional loads



Torsional loading

Load No.	Load type	Load (kNm)	Distance along beam (mm)
1	UDL	4.7	-

Design torque at LHS support; $T_A = T_{d\_1} / 2 = \mathbf{2.34}$ kNm

Design torque at RHS support; $T_B = T_{d\_1} / 2 = \mathbf{2.34}$ kNm

Average torque over haf the beam; $T_{av,t,Ed} = - T_{d\_1} / 4 = \mathbf{-1.17}$ kNm

Maximum St Venant torsion design moment; $T_{t,Ed}$ = max(abs(T_A), abs(T_B)) = **2.34** kNm

Rotation at mid-span; ϕ = abs($T_{avt,Ed}$) × L / (2 × G_{SEC3} × I_t) = **0.00123**

Additional minor axis moment; $M_{z,add,Ed} = \phi \times M_{y,Ed}$ = **0.06** kNm

Check shear - Section 6.2.6

Height of web; h_w = h - 2 × t = **284** mm

η = **1.000**

h_w / t = 35.5 = 38.4 × ε / η < 72 × ε / η

Shear buckling resistance can be ignored

Design shear force; $V_{y,Ed}$ = **38.10** kN

Shear area - cl 6.2.6(3); A_v = A × h / (b + h) = **4556** mm^2

Design shear resistance - cl 6.2.6(2); $V_{pl,y,Rd} = A_v \times (f_y / \sqrt{(3)}) / \gamma_{M0}$ = **723.4** kN

Shear stress due to St Venant torsion; $\tau_{t,Ed} = T_{t,Ed} / W_t$ = **6.06** N/mm^2

Reduced shear resistance due to torsion - eq 6.26; $V_{c,y,Rd} = V_{pl,T,y,Rd} = (1 - \tau_{t,Ed} / ((f_y / \sqrt{(3)}) / \gamma_{M0})) \times V_{pl,y,Rd}$ = **695.9** kN

$V_{y,Ed} / V_{pl,T,y,Rd}$ = **0.055**

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment; $M_{y,Ed}$ = **49.5** kNm

Design bending resistance moment - eq 6.13; $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **150.1** kNm

$M_{y,Ed} / M_{pl,y,Rd}$ = **0.33**

PASS - Design bending resistance moment exceeds design bending moment

Check bending moment - Section 6.2.5

Design bending moment; $M_{z,Ed,total} = M_{z,Ed} + M_{z,add,Ed}$ = **0.1** kNm

Design bending resistance moment - eq 6.13; $M_{c,z,Rd} = M_{pl,z,Rd} = W_{pl.z} \times f_y / \gamma_{M0}$ = **67.5** kNm

$M_{z,Ed,total} / M_{pl,z,Rd}$ = **0.001**

PASS - Design bending resistance moment exceeds design bending moment

Torsional resistance

Design resistance to St. Venant torsion; $T_{Rd} = C \times f_y / (\sqrt{(3)} \times \gamma_{M0})$ = **61.42** kNm

$T_{t,Ed} / T_{Rd}$ = **0.038**

PASS - Design torsional resistance exceeds applied St Venants torsion

Plastic verification - cl.6.2.9.1(6)

αbi = **1.66**

βbi = **1.66**

$[M_{y,Ed} / M_{pl,y,Rd}]^{\alpha bi} + [M_{z,Ed,total} / M_{pl,z,Rd}]^{\beta bi}$ = **0.159**

PASS - Plastic interaction criterion is less than 1.0

Serviceability limit checks

Rotation limit; $\phi_{ser,lim}$ = **3.00** deg

Rotation of torsion beam; $\phi_{ser} = M_{z,add,Ed} \times 180 / (M_{y,Ed} \times \gamma_G \times \pi)$ = **0.05** deg

PASS - Rotation limit exceeds rotation in torsion beam

Vertical deflection limit; $\delta_{v,lim}$ = **10.0** mm

SLS loading on beam; $f_{d,ser} = w_{sb,ser} + w_{tb,ser} + w_{sw,sb,ser} + w_{sw,tb,ser}$ = **10.61** kN/m

Vertical deflection of torsion beam; $\delta_v = 5 \times f_{d,ser} \times L^4 / (384 \times E_{sb} \times I_{ytb})$ = **7.6** mm

PASS - Vertical deflection limit exceeds vertical deflection in torsion beam

B1 MASONRY BEARING DESIGN (EN1996)

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	35.7 kN	35.8 kN	N/A	N/A	0.997	Pass

Masonry panel details

Panel length; L = **1500** mm

Panel height; h = **2500** mm

Thickness of load bearing leaf; t = **100** mm

Effective height; h_{ef} = **2500** mm

Effective thickness; t_{ef} = **100** mm

Masonry material details

Unit type; **Aggregate concrete - 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; $f_b = f_c \times k \times d_{sf}$ = **10.07** N/mm^2

Specific weight of units; γ = **18** kN/m^3

Mortar type; **M4 - General Purpose**

Compressive strength of mortar; f_m = **4.0** N/mm^2

Compressive strength factor - Tbl. NA 4; K = **0.75**

Characteristic compressive strength - eq. 3.1; $f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **5.73** N/mm^2

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

Modulus of elasticity - cl.3.7.2; $E_w = K_E \times f_k$ = **5727** N/mm^2

Design compressive strength of masonry

Category of manufacturing control; **Category II**

Class of execution control; **Class 2**

Partial factor for compressive strength; γ_M = **3.00**

Cross-sectional area of wall; A = L × t = **0.15** m^2;

Design compressive strength of masonry; $f_d = f_k / \gamma_M$ = **1.91** N/mm^2

Partial safety factors for design loads

Partial safety factor for permanent load; γ_{fG} = **1.35**

Partial safety factor for variable load; γ_{fQ} = **1.50**

Superimposed vertical loading details

Permanent UDL at top of wall; g_k = **0.00** kN/m

Variable UDL at top of wall; q_k = **0.00** kN/m

Eccentricity of permanent UDL load; e_{gu} = **0** mm

Eccentricity of variable UDL load; e_{qu} = **0** mm

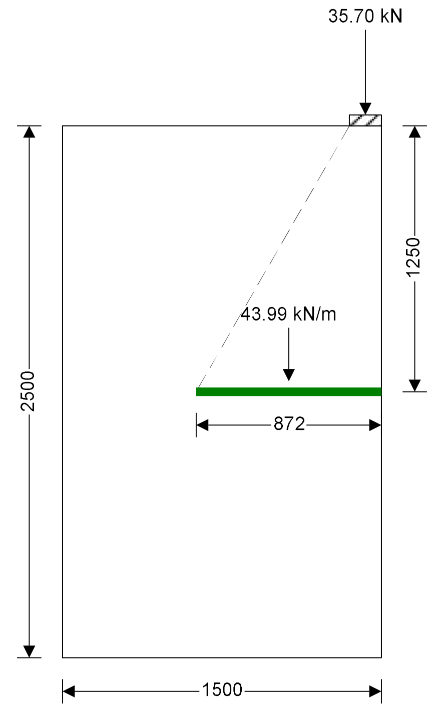
Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit; λ_{lim} = **27**

Slenderness ratio; $\lambda = h_{ef} / t_{ef}$ = **25.0**

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details - B1 point load



Permanent concentrated load; G_{kc1} = **20.00** kN

Variable concentrated load; Q_{kc1} = **5.80** kN

Eccentricity of concentrated load; e_{c1} = **0** mm

Length of concentrated load; L_{c1} = **150** mm

Width of concentrated load; w_{c1} = **100** mm

Height of concentrated load; h_{c1} = **2500** mm

Distance of load to right vertical edge; r_{11} = **0** mm

Distance of load to nearest vertical edge; a_{11} = **0** mm

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check; $e_{c1} <= t / 4$

PASS - Eccentricity of load is less than t/4

Area of bearing; $A_{b1} = L_{c1} \times w_{c1}$ = **15000** mm^2

Effective length of bearing at mid-height; $l_{efm1} = L_{c1} + h_{c1} / 2 \times \tan(30) + r_{11}$ = **872** mm

Effective bearing area; $A_{ef1} = l_{efm1} \times t$ = **87169** mm^2

Bearing area ratio check; $A_{ratio1} = \mathrm{Min}(A_{b1} / A_{ef1}, 0.45)$ = **0.17**

Initial enhancement factor; $\beta_{init1} = \mathrm{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0)$ = **1.31**

Maximum enhancement factor; $\beta_{max1} = \mathrm{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5)$ = **1.25**

Enhancement factor for concentrated loads; $\beta_1 = \mathrm{Min}(\beta_{init1}, \beta_{max1})$ = **1.25**

Design value of the concentrated load; $N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ}$ = **35.70** kN

Design value concentrated load resistance; $N_{Rdc1} = \beta_1 \times A_{b1} \times f_d$ = **35.79** kN

PASS - Design resistance exceeds applied concentrated load

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h)$ = **0.0** mm

Eccentricity of variable UDL at mid-height below concentrated load

$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h)$ = **0.0** mm

Eccentricity of concentrated load at mid-height; $e_{mc1} = e_{c1} / 2$ = **0.0** mm

Initial eccentricity - cl.5.5.1.1(4); $e_{init} = h_{ef} / 450$ = **5.6** mm

Concentrated load at mid-height as UDL; $N_{mc1} = N_{Edc1} / l_{efm1}$ = **40.96** kN/m

Vertical load at mid-height; $N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1}$ = **43.99** kN/m

Design moment at mid-height; $M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1}$ = **0.00** kNm/m

Eccentricities due to loads - eq. 6.7; $e_{m1} = \mathrm{Abs}(M_{Ed1}) / N_{Ed1} + e_{init}$ = **5.6** mm

Slenderness ratio limit for creep eccentricity; λ_c = **27**

Eccentricity due to creep; e_{k1} = **0.0** mm

Eccentricity at mid-height - eq. 6.6; $e_{mk1} = \mathrm{Max}(e_{m1} + e_{k1}, 0.05 \times t)$ = **5.6** mm

From eq. G2; $A_{11} = 1 - 2 \times e_{mk1} / t$ = **0.89**

From eq. G3; $u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t)$ = **1.09**

Capacity reduction factor - eq. G1; $\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2)$ = **0.49**

Design vertical resistance of panel - eq.6.2; $N_{Rd1} = \Phi_{m1} \times t \times f_d$ = **93.26** kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

<u>Provide 300x100x8.0 RHS S.355 with 8mm thick plate welded to underside. 150mm bearing length on 215lg x 215dp x 100thk concrete padstone.</u>

Beam B2 Design:

Loadings

UDL from Pitched Roof

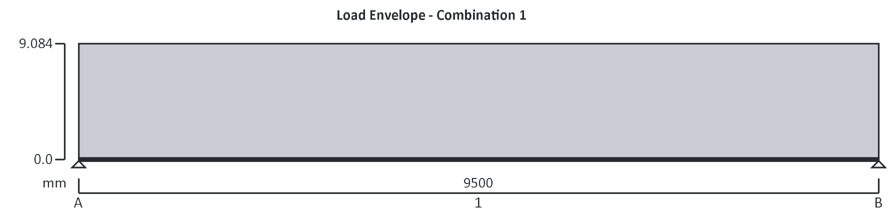
Width of roof being carried;	x_1 = **4.9** m		
Dead Load;	$DL_1 = 0.572 \text{ kN/m}^2 \times x_1$	=	**2.803** kN/m
Imposed Load;	$IL_1 = 0.60 \text{ kN/m}^2 \times x_1$	=	**2.940** kN/m

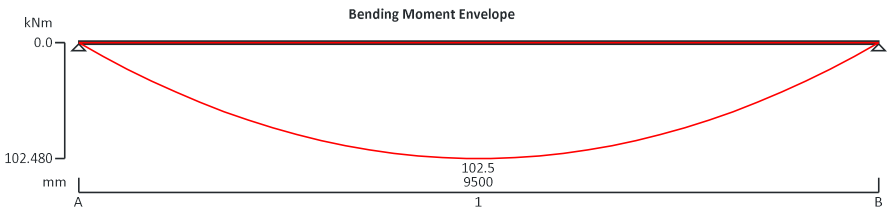
STEEL BEAM B2 ANALYSIS & DESIGN (EN1993)

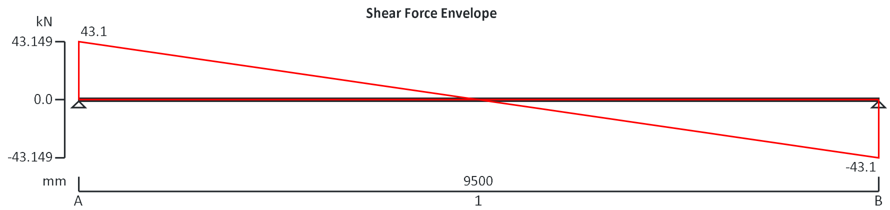
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

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







Support conditions

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

Applied loading

Beam loads Permanent self weight of beam × 1

Permanent full UDL 2.8 kN/m

Variable full UDL 2.94 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} = **102.5** kNm;	M_{min} = **0** kNm
Maximum shear;	V_{max} = **43.1** kN;	V_{min} = **-43.1** kN
Deflection;	δ_{max} = **13.2** mm;	δ_{min} = **0** mm
Maximum reaction at support A;	R_{A_max} = **43.1** kN;	R_{A_min} = **43.1** kN
Unfactored permanent load reaction at support A;	$R_{A\_Permanent}$ = **16.4** kN	
Unfactored variable load reaction at support A;	$R_{A\_Variable}$ = **14** kN	
Maximum reaction at support B;	R_{B_max} = **43.1** kN;	R_{B_min} = **43.1** kN
Unfactored permanent load reaction at support B;	$R_{B\_Permanent}$ = **16.4** kN	
Unfactored variable load reaction at support B;	$R_{B\_Variable}$ = **14** kN	

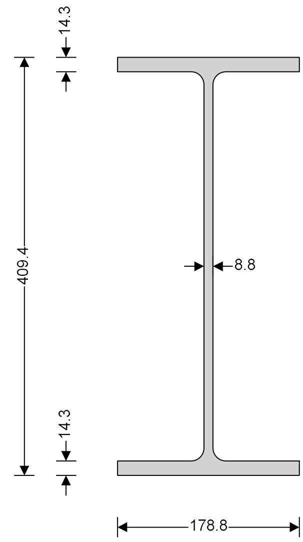
Section details

Section type; **UB 406x178x67 (British Steel Section Range 2022 (BS4-1))**

Steel grade; **S275**

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

Nominal thickness of element;	$t = \max(t_f, t_w)$ = **14.3** mm
Nominal yield strength;	f_y = **275** N/mm^2
Nominal ultimate tensile strength;	f_u = **410** N/mm^2
Modulus of elasticity;	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections; γ_{M0} = **1.00**

Resistance of members to instability; γ_{M1} = **1.00**

Resistance of tensile members to fracture; γ_{M2} = **1.10**

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis; K_y = **1.000**

Effective length factor in minor axis; K_z = **1.000**

Effective length factor for torsion; $K_{LT.A}$ = **1.000**;

$K_{LT.B}$ = **1.000**;

Classification of cross sections - Section 5.5

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

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

Width of section; c = d = **355.4** mm

$c / t_w = 43.7 \times \varepsilon <= 72 \times \varepsilon$; Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \times r) / 2$ = **72.3** mm

$c / t_f = 5.5 \times \varepsilon <= 9 \times \varepsilon$; Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \times t_f$ = **380.8** mm

Shear area factor; η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\mathrm{abs}(V_{max}), \mathrm{abs}(V_{min}))$ = **43.1** kN

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **3979** mm^2

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **631.7** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **102.5** kNm

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **372.7** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6; k_c = **0.94**

$C_1 = 1 / k_c^2$ = **1.132**

Curvature factor; $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.972**

Poissons ratio; ν = **0.3**

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

Unrestrained length; $L = 1.0 \times L_{s1}$ = **9500** mm

Elastic critical buckling moment; $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **147.2** kNm

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.591**

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

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

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5; c

Imperfection factor - Table 6.3; α_{LT} = **0.49**

Correction factor for rolled sections; β = **0.75**

LTB reduction determination factor; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **1.742**

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.356**

Modification factor; $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **1.000**

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.356**

Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **132.8** kNm

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection; $\delta_{lim} = L_{s1} / 360$ = **26.4** mm

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **13.196** mm

PASS - Maximum deflection does not exceed deflection limit

B2 MASONRY BEARING DESIGN (EN1996)

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	**43.1 kN**	**26.6 kN**	**1.31 N/mm²**	**1.48 N/mm²**	**0.885**	**Pass**

Masonry panel details

Panel length; L = **4000** mm

Panel height; h = **2500** mm

Thickness of load bearing leaf; t = **100** mm

Effective height; h_{ef} = **2500** mm

Effective thickness; t_{ef} = **100** mm

Masonry material details

Unit type; **Aggregate concrete - Group 1**

Compressive strength of masonry unit; f_c = **3.6** 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; $f_b = f_c \times k \times d_{sf}$ = **4.97** N/mm^2

Specific weight of units; γ = **18** kN/m^3

Mortar type; **M4 - General Purpose**

Compressive strength of mortar; f_m = **4.0** N/mm^2

Compressive strength factor - Tbl. NA 4; K = **0.75**

Characteristic compressive strength - eq. 3.1; $f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **3.49** N/mm^2

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

Modulus of elasticity - cl.3.7.2; $E_w = K_E \times f_k$ = **3491** N/mm^2

Design compressive strength of masonry

Category of manufacturing control; **Category II**

Class of execution control; **Class 2**

Partial factor for compressive strength; γ_M = **3.00**

Cross-sectional area of wall; $A = L \times t$ = **0.40** m^2;

Design compressive strength of masonry; $f_d = f_k / \gamma_M$ = **1.16** N/mm^2

Partial safety factors for design loads

Partial safety factor for permanent load; γ_{fG} = **1.35**

Partial safety factor for variable load; γ_{fQ} = **1.50**

Superimposed vertical loading details

Permanent UDL at top of wall; g_k = **0.00** kN/m

Variable UDL at top of wall; q_k = **0.00** kN/m

Eccentricity of permanent UDL load; e_{gu} = **0** mm

Eccentricity of variable UDL load; e_{qu} = **0** mm

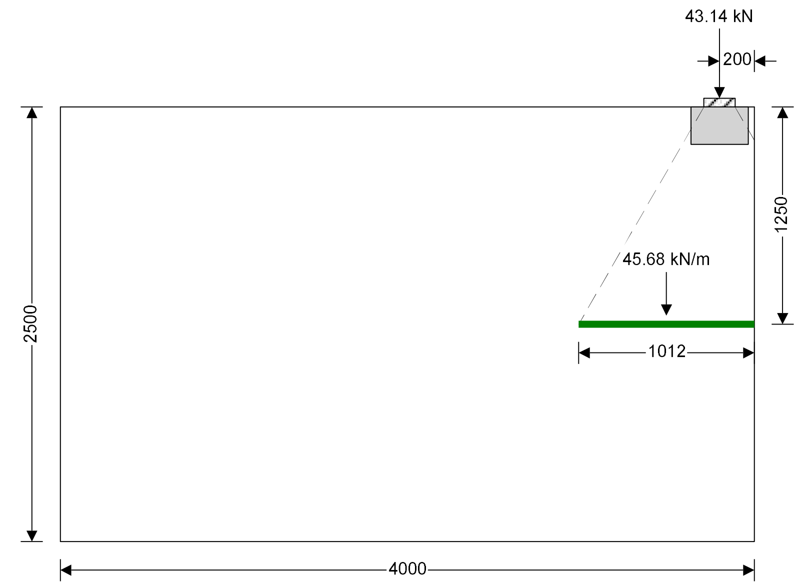
Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit; λ_{lim} = **27**

Slenderness ratio; $\lambda = h_{ef} / t_{ef}$ = **25.0**

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details - B2 point load



Permanent concentrated load; G_{kc1} = **16.40** kN

Variable concentrated load; Q_{kc1} = **14.00** kN

Eccentricity of concentrated load; e_{c1} = **0** mm

Length of concentrated load; L_{c1} = **180** mm

Width of concentrated load; w_{c1} = **100** mm

Height of concentrated load; h_{c1} = **2500** mm

Distance of load to right vertical edge; r_{11} = **110** mm

Distance of load to nearest vertical edge; a_{11} = **110** mm

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check; $e_{c1} <= t / 4$

PASS - Eccentricity of load is less than t/4

Area of bearing; $A_{b1} = L_{c1} \times w_{c1}$ = **18000** mm^2

Effective length of bearing at mid-height; $l_{efm1} = L_{c1} + h_{c1} / 2 \times \tan(30) + r_{11}$ = **1012** mm

Effective bearing area; $A_{ef1} = l_{efm1} \times t$ = **101169** mm^2

Bearing area ratio check; $A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45)$ = **0.18**

Initial enhancement factor; $\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0)$ = **1.32**

Maximum enhancement factor; $\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5)$ = **1.27**

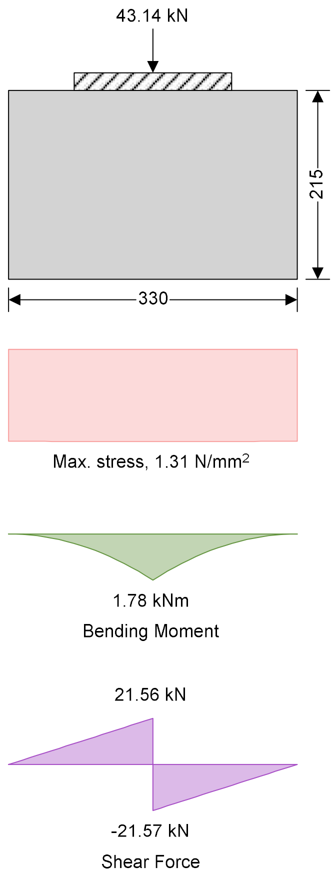
Enhancement factor for concentrated loads; $\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1})$ = **1.27**

Design value of the concentrated load; $N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ}$ = **43.14** kN

Design value concentrated load resistance; $N_{Rdc1} = \beta_1 \times A_{b1} \times f_d$ = **26.65** kN

Applied concentrated load exceeds design resistance, spreader required!

Design of spreader beam



Type of spreader; **Concrete padstone**

Type of bearing onto spreader; **Point load**

Location of load from RHS of spreader; P_{11} = **165** mm

Length of spreader; L_{sp1} = **330** mm

Height of spreader; h_{sp1} = **215** mm

Width of spreader; w_{sp1} = **100** mm

Eccentricity of load on spreader; e_{sp1} = **0** mm

Modulus of elasticity; E_{sp1} = **29962** N/mm^2

Second moment of area; $I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3$ = **82819792** mm^4

Modulus of the wall; $k_0 = E_w / h$ = **1.40** N/mm^2/mm

Winkler's constant; $K_{c1} = k_0 \times w_{sp1}$ = **139.66** N/mm/mm

Characteristic of the system; $\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4}$ = **0.00194** mm^{-1}

Classification of spreader; $\alpha L_1 = \alpha_1 \times L_{sp1}$ = **0.64**; **Medium**

Krilov's functions for the spreader length; $B_{\alpha l1} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi))$ = **0.64**

$C_{\alpha l1} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi)$ = **0.20**

$D_{\alpha l1} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi))$ = **0.04**

Krilov's functions at the point load; $A_{\alpha P11} = \cosh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi)$ = **1.00**

$B_{\alpha P11} = 1/2 \times (\cosh(\alpha_1 \times P_{11}) \times \sin(180 \times \alpha_1 \times P_{11} / \pi) + \sinh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi))$ = **0.32**

Using method of initial conditions

Initial moment of LH edge; M_{01} = **0** kNm

Initial shear of LH edge; V_{01} = **0** kN

Which gives; $(4 \times \alpha_1^2 \times C_{\alpha l1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha l1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - B_{\alpha P11} / \alpha_1 \times N_{Edc1}$ = **0.00** kNm

and; $(4 \times \alpha_1^3 \times B_{\alpha l1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha l1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - A_{\alpha P11} \times N_{Edc1}$ = **0.00** kN

Therefore,

Initial deflection of LH edge; δ_{01} = **0.93313** mm

Initial rotationof LH edge; Φ_{01} = **0.000039**

Location of maximum deflection; x_{def1} = **165** mm

Krilov's functions at the spreader length; $A_{\alpha xdef1} = \cosh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi)$ = **1.00**

$B_{\alpha xdef1} = 1/2 \times (\cosh(\alpha_1 \times x_{def1}) \times \sin(180 \times \alpha_1 \times x_{def1} / \pi) + \sinh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi))$ = **0.32**

Distance of point load right of loaction; p_{1def1} = **0** mm

Krilov's functions at the spreader length; $D_{\alpha p1def1} = 1/4 \times (\cosh(\alpha_1 \times p_{1def1}) \times \sin(180 \times \alpha_1 \times p_{1def1} / \pi) - \sinh(\alpha_1 \times p_{1def1}) \times \cos(180 \times \alpha_1 \times p_{1def1} / \pi))$ = **0.00**

Particular integral due to load; $\delta'_1 = D_{\alpha p1def1} / \alpha_1^3 \times N_{Edc1} / (I_{sp1} \times E_{sp1})$ = **0.000** mm

Maximum deflection; $\delta_{max1} = A_{\alpha xdef1} \times \delta_{01} + B_{\alpha xdef1} \times \Phi_{01} / \alpha_1 + \delta'_1$ = **0.938** mm

Location of maximum moment; x_{M1} = **165** mm

Krilov's functions at the spreader length; $C_{\alpha xM1} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi)$ = **0.05**

$D_{\alpha xM1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi))$ = **0.01**

Distance of point load right of loaction; p_{1M1} = **0** mm

Krilov's functions at the spreader length; $B_{\alpha p1M1} = 1/2 \times (\cosh(\alpha_1 \times p_{1M1}) \times \sin(180 \times \alpha_1 \times p_{1M1} / \pi) + \sinh(\alpha_1 \times p_{1M1}) \times \cos(180 \times \alpha_1 \times p_{1M1} / \pi))$ = **0.00**

Particular integral due to load; $M'_1 = -B_{\alpha p1M1} / \alpha_1 \times N_{Edc1}$ = **0.00** kNm

Maximum moment; $M_{Edsp1} = (4 \times \alpha_1^2 \times C_{\alpha xM1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha xM1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M'_1$ = **1.78** kNm

Location of maximum shear; x_{V1} = **165** mm

Krilov's functions at the spreader length; $B_{\alpha xV1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi))$ = **0.32**

$C_{\alpha xV1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi)$ = **0.05**

Distance of point load right of loaction; p_{1V1} = **0** mm

Krilov's functions at the spreader length; $A_{\alpha p1V1} = \cosh(\alpha_1 \times p_{1V1}) \times \cos(180 \times \alpha_1 \times p_{1V1} / \pi)$ = **1.00**

Particular integral due to load; $V'_1 = -A_{\alpha p1V1} \times N_{Edc1}$ = **-43.14** kN

Shear at concentrated point load; $V_1 = (4 \times \alpha_1^3 \times B_{\alpha xV1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha xV1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + V'_1$ = **-21.57** kN

Maximum shear; $V_{Edsp1} = \mathrm{Max}(\mathrm{Abs}(V_1), N_{Edc1} - \mathrm{Abs}(V_1))$ = **21.57** kN

Maximum allowable stress under spreader; $\sigma_{Rdsp1} = \beta_1 \times f_d$ = **1.48** N/mm^2

Maximum reaction; $N_{Edsp1} = K_{c1} \times \delta_{max1}$ = **131.00** kN/m

Design stress; $\sigma_{Edsp1} = N_{Edsp1} / w_{sp1}$ = **1.31** N/mm^2

PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$e_{gmu1} = e_{gu} \times h_{c1} / (2 \times h)$ = **0.0** mm

Eccentricity of variable UDL at mid-height below concentrated load

$e_{qmu1} = e_{qu} \times h_{c1} / (2 \times h)$ = **0.0** mm

Eccentricity of concentrated load at mid-height; $e_{mc1} = e_{c1} / 2$ = **0.0** mm

Initial eccentricity - cl.5.5.1.1(4); $e_{init} = h_{ef} / 450$ = **5.6** mm

Concentrated load at mid-height as UDL; $N_{mc1} = N_{Edc1} / l_{efm1}$ = **42.64** kN/m

Vertical load at mid-height; $N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1}$ = **45.68** kN/m

Design moment at mid-height; $M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1}$ = **0.00** kNm/m

Eccentricities due to loads - eq. 6.7; $e_{m1} = \mathrm{Abs}(M_{Ed1}) / N_{Ed1} + e_{init}$ = **5.6** mm

Slenderness ratio limit for creep eccentricity; λ_c = **27**

Eccentricity due to creep; e_{k1} = **0.0** mm

Eccentricity at mid-height - eq. 6.6; $e_{mk1} = \mathrm{Max}(e_{m1} + e_{k1}, 0.05 \times t)$ = **5.6** mm

From eq. G2; $A_{11} = 1 - 2 \times e_{mk1} / t =$ **0.89**

From eq. G3; $u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) =$ **1.09**

Capacity reduction factor - eq. G1; $\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) =$ **0.49**

Design vertical resistance of panel - eq.6.2; $N_{Rd1} = \Phi_{m1} \times t \times f_d =$ **56.86** kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

<u>Provide 406x178x67 UB S.275 on 330lg x 215dp x 100thk concrete padstones.</u>

Beam TB1 Design:

Loadings

UDL from Pitched Roof

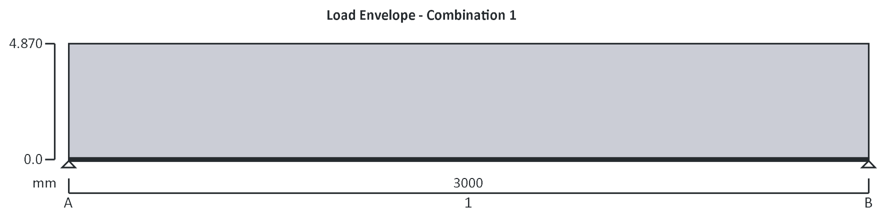
Width of roof being carried;	x_1 = **2.85** m		
Dead Load;	$DL_1 = 0.572\ kN/m^2 \times x_1$	=	**1.630** kN/m
Imposed Load;	$IL_1 = 0.60\ kN/m^2 \times x_1$	=	**1.710** kN/m

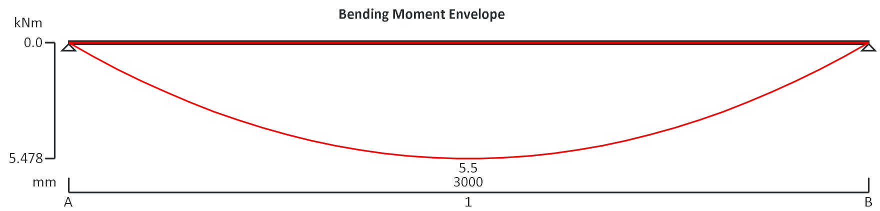
TIMBER BEAM TB1 ANALYSIS & DESIGN (EN1995)

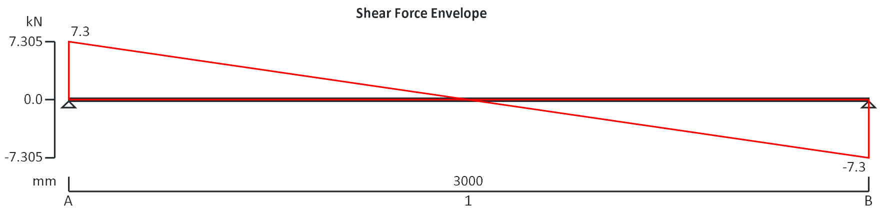
TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.05







Applied loading

Beam loads

Permanent self weight of beam × 1

Permanent full UDL 1.630 kN/m

Variable full UDL 1.710 kN/m

Load combinations

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

Analysis results

Maximum moment; M_{max} = **5.478** kNm; M_{min} = **0.000** kNm

Design moment; M = max(abs(M_{max}),abs(M_{min})) = **5.478** kNm

Maximum shear; F_{max} = **7.305** kN; F_{min} = **-7.305** kN

Design shear; F = max(abs(F_{max}),abs(F_{min})) = **7.305** kN

Total load on beam; W_{tot} = **14.609** kN

Reactions at support A; R_{A_max} = **7.305** kN; R_{A_min} = **7.305** kN

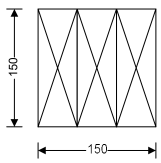
Unfactored permanent load reaction at support A; $R_{A\_Permanent}$ = **2.561** kN

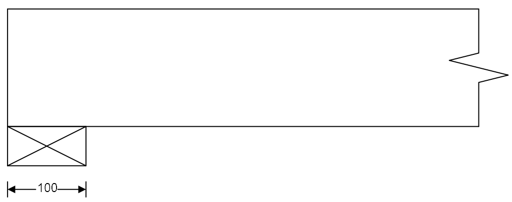
Unfactored variable load reaction at support A; $R_{A\_Variable}$ = **2.565** kN

Reactions at support B; R_{B_max} = **7.305** kN; R_{B_min} = **7.305** kN

Unfactored permanent load reaction at support B; $R_{B\_Permanent}$ = **2.561** kN

Unfactored variable load reaction at support B; $R_{B\_Variable}$ = **2.565** kN





Timber section details

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

Depth of timber sections; h = **150** mm

Number of timber sections in member; N = **3**

Overall breadth of timber member; $b_b = N \times b$ = **150** mm

Timber strength class - EN 338:2016 Table 1; **C24**

Member details

Load duration - cl.2.3.1.2; **Medium-term**

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

Length of span; L_{s1} = **3000** mm

Length of bearing; L_b = **100** mm

Section properties

Cross sectional area of member; $A = N \times b \times h$ = **22500** mm^2

Section modulus; $W_y = N \times b \times h^2 / 6$ = **562500** mm^3

$W_z = h \times (N \times b)^2 / 6$ = **562500** mm^3

Second moment of area; $I_y = N \times b \times h^3 / 12 = \mathbf{42187500}$ mm^4

$I_z = h \times (N \times b)^3 / 12 = \mathbf{42187500}$ mm^4

Radius of gyration; $r_y = \sqrt{(I_y / A)} = \mathbf{43.3}$ mm

$r_z = \sqrt{(I_z / A)} = \mathbf{43.3}$ mm

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3; $\gamma_M = \mathbf{1.300}$

Modification factors

Modification factor for load duration and moisture content - Table 3.1

$k_{mod} = \mathbf{0.800}$

Deformation factor for service classes - Table 3.2; $k_{def} = \mathbf{0.600}$

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

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

Bending stress re-distribution factor - cl.6.1.6(2); $k_m = \mathbf{0.700}$

Crack factor for shear resistance - cl.6.1.7(2); $k_{cr} = \mathbf{0.670}$

Load configuration factor - exp.6.4; $k_{c.90} = \mathbf{1.000}$

System strength factor - cl.6.6; $k_{sys} = \mathbf{1.000}$

Lateral buckling factor - cl.6.3.3(5); $k_{crit} = \mathbf{1.000}$

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress; $\sigma_{c.90.d} = R_{A\_max} / (N \times b \times L_b) = \mathbf{0.487}$ N/mm^2

Design compressive strength; $f_{c.90.d} = k_{mod} \times k_{sys} \times k_{c.90} \times f_{c.90.k} / \gamma_M = \mathbf{1.538}$ N/mm^2

$\sigma_{c.90.d} / f_{c.90.d} = \mathbf{0.317}$

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress; $\sigma_{m.d} = M / W_y = \mathbf{9.740}$ N/mm^2

Design bending strength; $f_{m.d} = k_{h.m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m.k} / \gamma_M = \mathbf{14.769}$ N/mm^2

$\sigma_{m.d} / f_{m.d} = \mathbf{0.659}$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress; $\tau_d = 3 \times F / (2 \times k_{cr} \times A) = \mathbf{0.727}$ N/mm^2

Permissible shear stress; $f_{v.d} = k_{mod} \times k_{sys} \times f_{v.k} / \gamma_M = \mathbf{2.462}$ N/mm^2

$\tau_d / f_{v.d} = \mathbf{0.295}$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit; $\delta_{lim} = \min(14 \text{ mm}, 0.004 \times L_{s1}) = \mathbf{12.000}$ mm

Instantaneous deflection due to permanent load; $\delta_{instG} = \mathbf{4.029}$ mm

Final deflection due to permanent load; $\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = \mathbf{6.446}$ mm

Instantaneous deflection due to variable load; $\delta_{instQ} = \mathbf{4.035}$ mm

Factor for quasi-permanent variable action; $\psi_2 = \mathbf{0.3}$

Final deflection due to variable load; $\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def})$ = **4.761** mm

Total final deflection; $\delta_{fin} = \delta_{finG} + \delta_{finQ}$ = **11.207** mm

$\delta_{fin} / \delta_{lim}$ = **0.934**

PASS - Total final deflection is less than the deflection limit

Provide 3No. 50x150 C24 bolted together with M12 at 400c/c.