



Project Preface

<i>Project name:</i>	Calculations for the steel beams and retaining wall
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<i>Client name:</i>	Angharad Davies
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<u><i>Construction site address:</i></u> <i>House no, Street:</i> <i>City/Town:</i> <i>Postcode:</i>	13 Tyntaldwyn Road, Merthyr Tydfil, CF48 4NG
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<i>Document prepared by:</i>	Marcin Krawucki BSc (Hons) MCI OB, MIET, ACABE, GMICE MCI OB Membership no : 7006632 MIET Membership no : 1101281027 ACABE Membership no : 80455303 GMICE Membership no : 92007313
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<i>Address:</i>	50 Quarry Row Merthyr Tydfil, CF47 8PS
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<i>Date of preparation:</i>	30-10-2025
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<i>Job reference number:</i>	AUX609
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<i>Revision:</i>	C01
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Design content

1 Introduction

1.1 Instructions

In accordance with instructions received from the client I have carried out a Structural calculations for the steel beams and retaining wall at

**13 Tyntaldwyn Road,
Merthyr Tydfil,
CF48 4NG**

2 Construction works

All the construction works must be done by experienced builder or contractor.

Before the works starts all the unnecessary loading from the slabs above which sits on the wall with the proposed opening must be removed.

Acrow No. 2 (SWL for 2.75m is 23kN and for 3.00m is 19kN per one prop) or similar.

Please contact supplier for double checking. Then the props can be installed with spacing not more than 1.0m apart. Then the existing lintels can be removed one by one and replacing them with steel beams. During installation, the beam might need to be temporary supported.

After confirming that the beams are installed properly the Acrow props can be slowly removed.

Please ensure that nothing unusual is happening during this process.

I also checked the maximum loading from the existing wall together with the reaction from the proposed beam. I assume that the foundation width is 900mm therefore the maxlimum pressure on the foundation is around 100kN/m² which is satisfactory.

The beam sizes are **shown in the drawings and are all in S275 grade.**

If the builder/contractor has any queries regarding the above-mentioned works during the installation installation process, please contact me.

All sizes of the beams and padstones are shown on the drawings

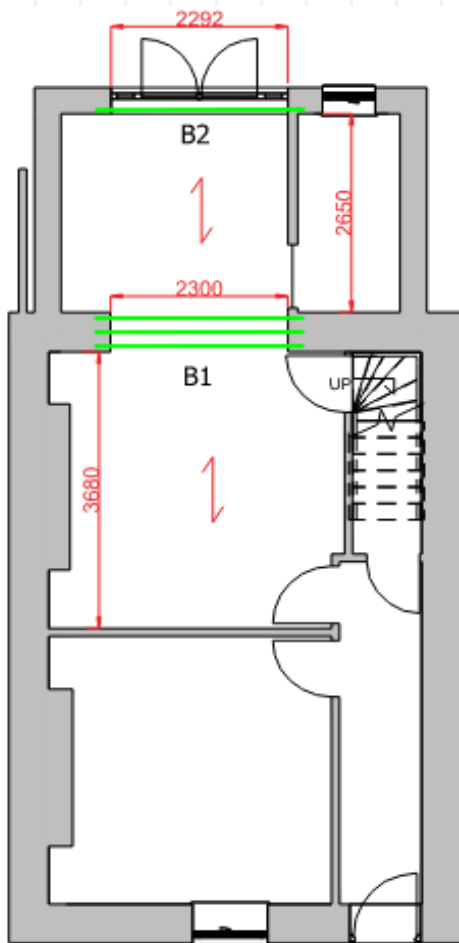
For more complex project a separate design for temporary works might be required.

Marcin Krawucki
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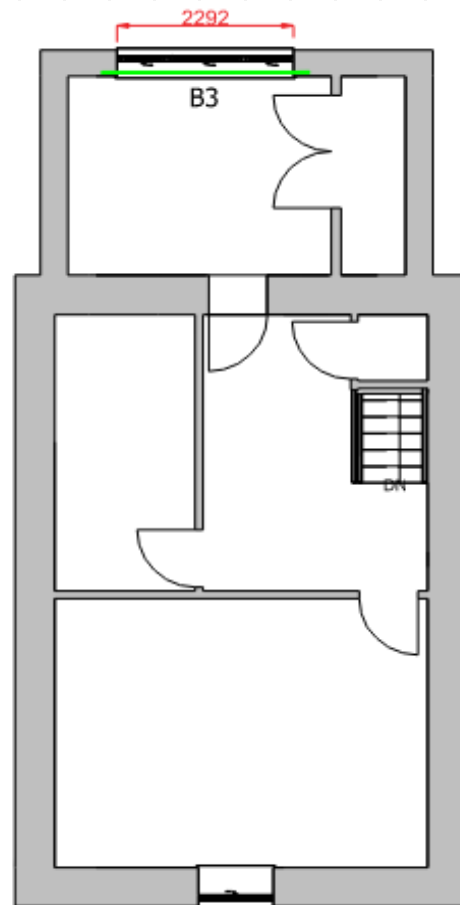


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Ground floor and first floor plans with beam location



GROUND FLOOR PLAN SHOWS
STRUCTURE ABOVE



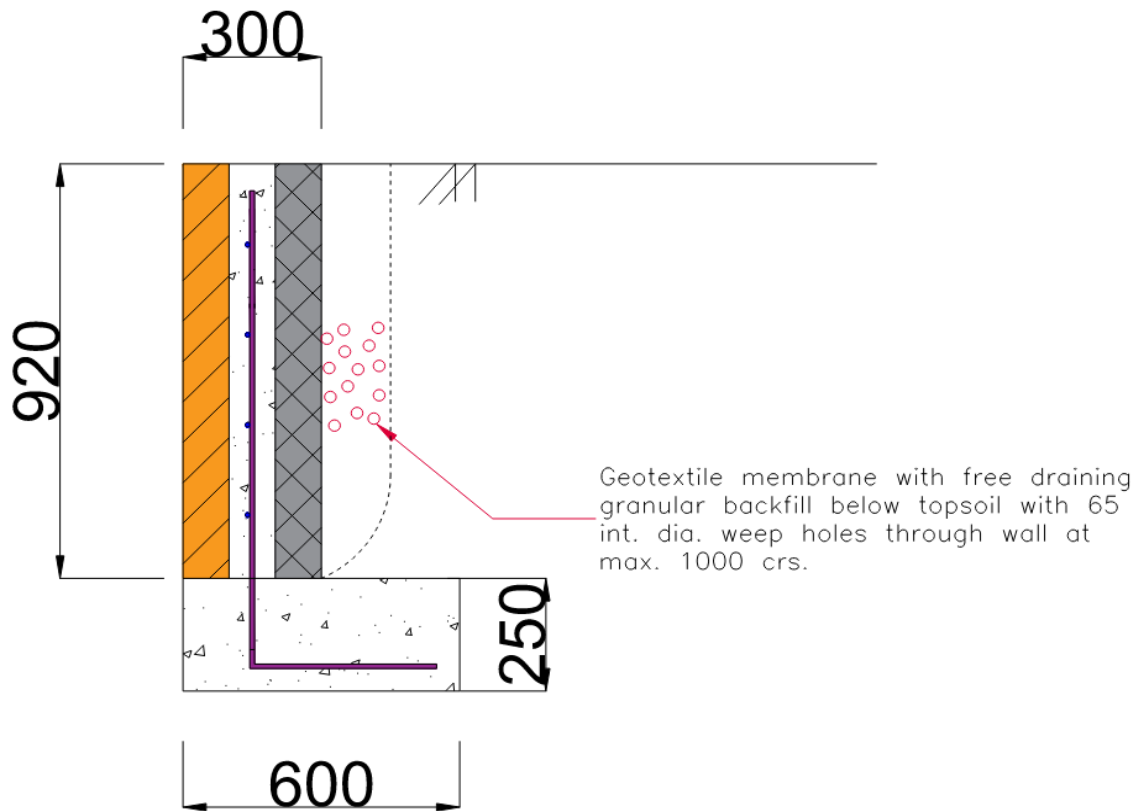
FIRST FLOOR PLAN SHOWS
STRUCTURE ABOVE

* padstone must be long enough to overlap blockwork from the blocked opening to spread the loading on longer length of the foundation



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Retaining wall





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Calculation for: **Loadings on beams**

LOAD TAKEDOWN

B1

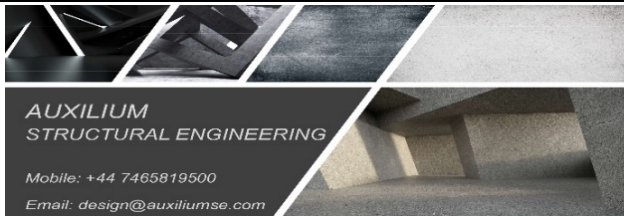
Wall				
DL	10.00kN/m ² x	2.80m	=	28.00kN/m
FIRST FLOOR				
DL	0.50kN/m ² x	3.20m	=	1.60kN/m
LL	2.00kN/m ² x	3.20m	=	6.40kN/m
Pitched Roof				
DL	0.85kN/m ² x	2.60m	=	2.21kN/m
LL	0.75kN/m ² x	2.60m	=	1.95kN/m
Total dead UDL				= 31.81kN/m
Total Live UDL				= 8.35kN/m

B2

Wall				
DL	4.00kN/m ² x	1.60m	=	6.40kN/m
First floor				
DL	0.50kN/m ² x	1.40m	=	0.70kN/m
LL	2.00kN/m ² x	1.40m	=	2.80kN/m
Total dead UDL				= 7.10kN/m
Total Live UDL				= 2.80kN/m

B3

Wall				
DL	4.00kN/m ² x	1.60m	=	6.40kN/m
Pitched Roof Distributed load				
DL			=	2.00kN/m
LL			=	1.80kN/m
Total dead UDL				= 8.40kN/m
Total Live UDL				= 1.80kN/m



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Calculation for: **Loadings on foundations under beams**

Loadings from Lintels B1 on Rear wall

Beams DL 36.6 kN LL 10 kN

Total loading 46.6 kN

Total loading from the beam B01 over 1.80m 46.60kN / 1.80m = 25.89kN/m

Loading from the existing wall (5m avg) = 50.0 kN/m

Loading from the existing roof and floor 12.0 kN/m

Total: 87.9 kN/m

Foundation width: 0.90 m

Pressure on ground: 97.7 kN/m²

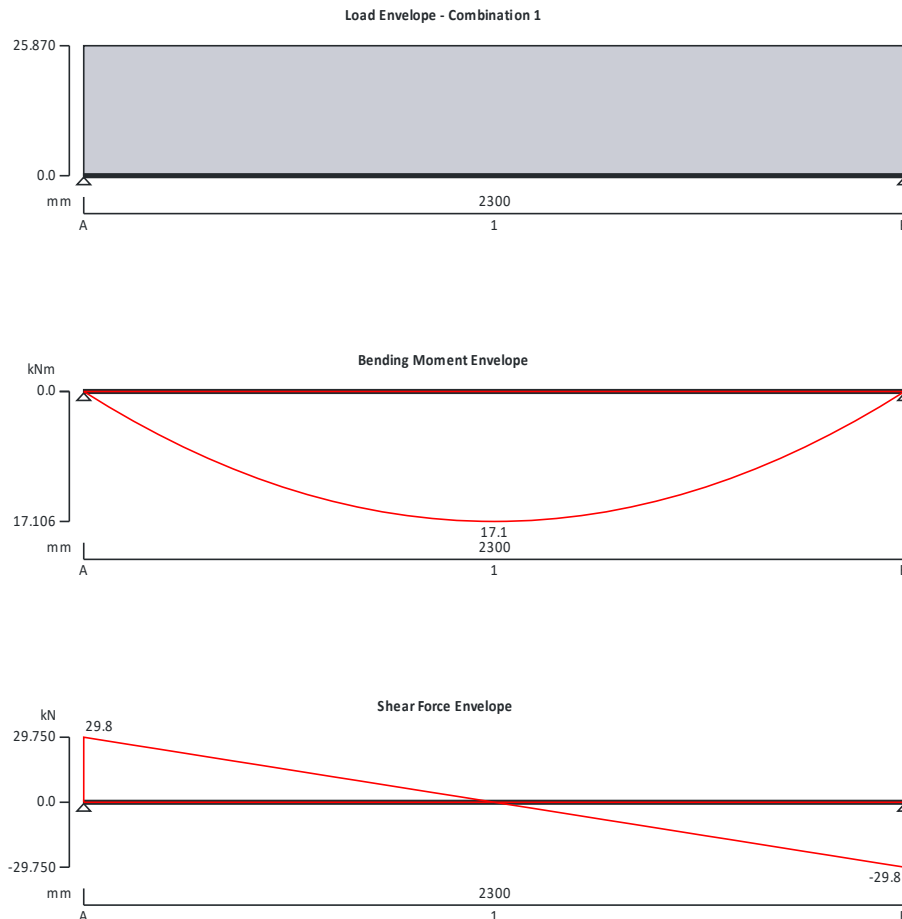
Satisfactory

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Section 1 B1				Sheet no./rev. 01/ 1	
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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 3.0.15



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 12.66 kN/m

Variable full UDL 5.65 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

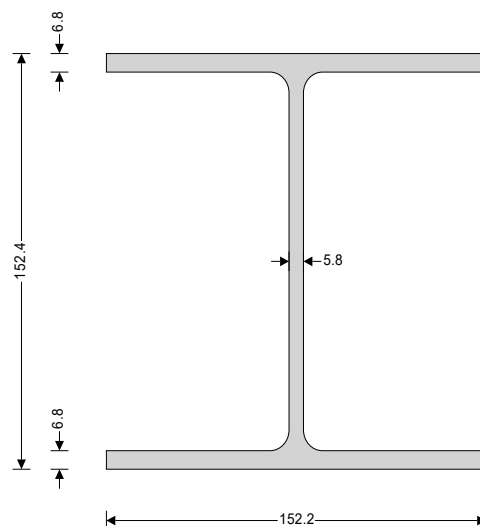
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Analysis results

Maximum moment	$M_{\max} = 17.1 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 29.8 \text{ kN}$	$V_{\min} = -29.8 \text{ kN}$
Deflection	$\delta_{\max} = 2.6 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 29.8 \text{ kN}$	$R_{A_{\min}} = 29.8 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 14.8 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 6.5 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 29.8 \text{ kN}$	$R_{B_{\min}} = 29.8 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 14.8 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 6.5 \text{ kN}$	

Section details

Section type	UC 152x152x23 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_f, t_w) = 6.8 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis	$K_y = 1.000$
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Effective length factor in minor axis

$$K_z = 1.000$$

Effective length factor for torsion

$$K_{LTA} = 1.000$$

$$K_{LTB} = 1.000$$

Classification of cross sections - Section 5.5

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

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

Width of section

$$c = d = 123.6 \text{ mm}$$

$$c / t_w = 23.1 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 65.6 \text{ mm}$$

$$c / t_f = 10.4 \times \varepsilon \leq 14 \times \varepsilon \quad \text{Class 3}$$

Section is class 3

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 138.8 \text{ mm}$$

Shear area factor

$$\eta = 1.000$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 29.8 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 997 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 158.4 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

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

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_{\max}}), \text{abs}(M_{s1_{\min}})) = 17.1 \text{ kNm}$$

Design bending resistance moment - eq 6.14

$$M_{c,Rd} = M_{el,Rd} = W_{el,y} \times f_y / \gamma_{M0} = 45.1 \text{ kNm}$$

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

$$g = \sqrt{[1 - (I_z / I_y)]} = 0.825$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$$

Unrestrained length

$$L = 1.0 \times L_{s1} = 2300 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 188.6 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{el,y} \times f_y / M_{cr})} = 0.489$$

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

$$b$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = 0.34$$

Correction factor for rolled sections

$$\beta = 0.75$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.605$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.965$$

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Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.976}$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.989}$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{el,y} \times f_y / \gamma_{M1} = \mathbf{44.6 \text{ kNm}}$

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 $\delta_{lim} = L_{s1} / 360 = \mathbf{6.4 \text{ mm}}$

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{2.573 \text{ mm}}$

PASS - Maximum deflection does not exceed deflection limit

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STEEL MASONRY SUPPORT

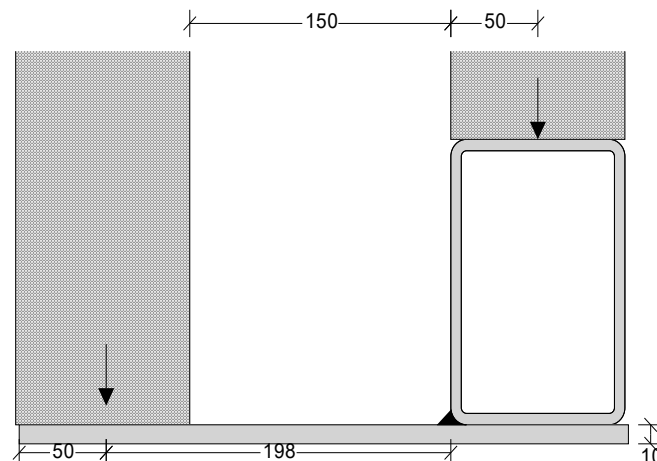
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 1.0.06

Design summary

Overall design status **PASS**
Overall design utilisation **0.784**

Description	Unit	Allowable	Applied	Utilisation	Result
Heel moment	kNm/m	4.583	0.962	0.210	PASS
Deflection	mm	1.7	0.7	0.444	PASS
Weld capacity	kN/m	1071.2	839.7	0.784	PASS
Shear force (major axis)	kN	251.9	17.5	0.070	PASS
Bending (major axis)	kNm	38.7	10.1	0.261	PASS
Bending (minor axis)	kNm	29.1	0.0	0.000	PASS
Torsion resistance	kNm	23.4	1.4	0.061	PASS
Plastic interaction				0.107	PASS
Torsion beam rotation	deg	3.00	0.05	0.015	PASS
Torsion beam deflection	mm	10.0	2.2	0.223	PASS



Partial factors - Section 6.1

Resistance of cross-sections	$\gamma_{M0} = 1$
Resist. of members to instability	$\gamma_{M1} = 1$
Resistance of joints	$\gamma_{M2} = 1.25$
Partial factor for permanent action	$\gamma_G = 1.35$
Partial factor for variable action	$\gamma_Q = 1.50$

Steel beam section details

Torsion beam section type	RHS 150x100x6.0
Nominal yield strength	$f_y = f_{y,tb} = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = f_{u,tb} = 410 \text{ N/mm}^2$

Masonry support section details

Section type	Plate 350x10(248)
Steel grade	User defined
Nominal thickness	$t_{nom,sb} = t_{plate} = 10 \text{ mm}$

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Nominal yield strength $f_{y, sb} = 275 \text{ N/mm}^2$
 Nominal ultimate tensile strength $f_{u, sb} = 410 \text{ N/mm}^2$
 Modulus of elasticity $E_{sb} = 210000 \text{ N/mm}^2$
 Total length of plate $l_{plate} = 350 \text{ mm}$
 Length of plate beyond outer edge of torsion beam $l_h = 248 \text{ mm}$

Supported materials detail

Density of masonry on torsion beam $\rho_{m, tb} = 20.0 \text{ kN/m}^3$
 Width of masonry on torsion beam $b_{m, tb} = 100 \text{ mm}$
 Height of masonry on torsion beam $h_{m, tb} = 1600 \text{ mm}$
 Eccentricity of torsion beam masonry $e_{load, tb} = 50 \text{ mm}$
 Eccentricity of torsion beam material $e_{tb} = 0 \text{ mm}$
 Add perm. force torsion beam (not masonry) $G_{k, add, tb} = 2.0 \text{ kN/m}$
 Add var. force torsion beam (not masonry) $Q_{k, add, tb} = 1.8 \text{ kN/m}$
 Density of masonry on support beam $\rho_{m, sb} = 20.0 \text{ kN/m}^3$
 Width of masonry on support beam $b_{m, sb} = 100 \text{ mm}$
 Height of masonry on support beam $h_{m, sb} = 1800 \text{ mm}$
 Eccentricity of support beam masonry $e_{load, sb} = 198 \text{ mm}$

Geometry

Cavity width $b_{cavity} = 150 \text{ mm}$
 Supported width of masonry $d_m = l_h + t_{shim} + e_{tb} - b_{cavity} = 98 \text{ mm}$

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

Maximum overall bending moment $M_{y, Ed} = 10.1 \text{ kNm}$
 Dist to NA combined section (CoG torsion beam) $Z_{na, all} = (h_{tb} + t_{plate}) \times A_{pl} / (2 \times (A_{tb} + A_{pl})) = 44 \text{ mm}$
 Second moment of area of combined section $I_{y, all} = (I_{y, tb} + A_{tb} \times Z_{na, all}^2) + A_{pl} \times (h_{tb} / 2 + t_{plate} / 2 - Z_{na, all})^2 = 1861 \text{ cm}^4$
 Elastic section modulus of combined section $Z_{y, all} = I_{y, all} / (h_{tb} / 2 + t_{plate} - Z_{na, all}) = 457.57 \text{ cm}^3$
 Section modulus of plate $Z_{y, plate} = 1m \times t_{plate}^2 / (6 \times 1m) = 16.67 \text{ cm}^3/m$
 Force of masonry on support plate $F_1 = (b_{m, sb} \times h_{m, sb} \times \rho_{m, sb} + G_{k, add, sb}) \times \gamma_G + Q_{k, add, sb} \times \gamma_Q = 4.9 \text{ kN/m}$
 Bending at heel $M_{y, Ed, plate} = F_1 \times e_{load, sb} = 1.0 \text{ kNm/m}$
 Moment capacity of plate $M_{y, Rd, plate} = Z_{y, plate} \times f_{y, sb} / \gamma_{M0} = 4.6 \text{ kNm/m}$

PASS - Moment capacity of plate exceeds applied moment

Longitudinal stress due to overall bending $\sigma_1 = M_{y, Ed} / Z_{y, all} = 22.0 \text{ N/mm}^2$
 Constant relating to Von Mises curve $C_{fp} = (4 \times f_{y, sb}^2 - 3 \times \sigma_1^2)^{0.5} = 548.7 \text{ N/mm}^2$
 Transverse bending stress ratio limit $\alpha_{ts} = (C_{fp}^2 - \sigma_1^2) / (2 \times C_{fp} \times f_{y, sb}) = 0.996$
 Transverse bending stress ratio $\alpha_{ts} = M_{y, Ed, plate} / M_{y, Rd, plate} = 0.210$

PASS - Transverse bending stress ratio less than allowable limit

Deflection of plate

Unfactored force on support angle $F_{1ser} = (b_{m, sb} \times h_{m, sb} \times \rho_{m, sb} + G_{k, add, sb}) + Q_{k, add, sb} = 3.6 \text{ kN/m}$
 Distance from weld to load position $a_m = e_{load, sb} = 198 \text{ mm}$
 Length of load resultant to edge of plate $b_m = l_h - e_{load, sb} = 50 \text{ mm}$
 Dist from weld to load position as ratio of length $a_l = a_m / (a_m + b_m) = 0.798$
 Effective second moment of area $I_{eff, def} = t_{plate}^3 / 12 = 83333 \text{ mm}^4/m$
 Deflection at end of plate $\delta = (a^2 \times (3 - a) / 6) \times (F_{1ser} \times (a_m + b_m)^3) / (E_{sb} \times I_{eff, def}) = 0.73 \text{ mm}$
 Deflection limit $\delta_{lim} = \min((1 + d_m / b_{cavity}) \times 1\text{mm}, 2\text{mm}) = 1.65 \text{ mm}$

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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	$F_A = F_1 \times \max((1 + (3 \times e_{load, sb}) / (2 \times b_{tb} / 2)), 1.4) = 33.7 \text{ kN/m}$
Maximum possible force in plate	$F_p = (l_h + \min(b_{tb}, l_{plate} - l_h)) \times t_{plate} \times f_{y, sb} = 957.0 \text{ kN}$
Longitudinal shear between beam and plate	$F_l = 2 \times F_p / L = 832.2 \text{ kN/m}$
Horizontal shear between beam and plate	$F_h = F_1 \times e_{load, sb} / (s_{weld} / 2 + t_{plate} / 2) = 106.9 \text{ kN/m}$
Resultant weld force	$F_{w, Ed} = (F_A^2 + F_l^2 + F_h^2)^{0.5} = 839.7 \text{ kN/m}$
Leg length of weld	$s_{weld} = 8.00 \text{ mm}$
Throat thickness of weld	$a_{weld} = 1 / \sqrt{2} \times s_{weld} = 5.66 \text{ mm}$
Length of weld per metre run	$l_{weld} = 1000 \text{ mm/m}$
Ultimate tensile strength used for weld	$f_{u, weld} = \min(f_{u, sb}, f_{u, tb}) = 410.0 \text{ N/mm}^2$
Correlation factor (table 4.1)	$\beta_w = 1.00$
Design shear strength	$f_{vw, d} = f_{u, weld} / (\sqrt{3} \times \beta_w \times \gamma_{M2}) = 189.4 \text{ N/mm}^2$
Design resistance of weld	$F_{w, Rd} = f_{vw, d} \times a_{weld} = 1071.2 \text{ kN/m}$

PASS - weld capacity exceeds applied force

Eccentricities

Distance to shear centre of torsion beam	$e_{0, tb} = 0 \text{ mm}$
Eccentricity of support beam masonry	$e_{m, sb} = e_{load, sb} + b_{tb} / 2 = 248 \text{ mm}$
Eccentricity of torsion beam masonry	$e_{m, tb} = b_{tb} / 2 - e_{load, tb} = 0 \text{ mm}$
Eccentricity of support beam	$e_{b, sb} = C_{zsb} + b_{tb} / 2 = 123 \text{ mm}$
Eccentricity of torsion beam	$e_{b, tb} = 0 \text{ mm}$

Torsional loading ULS

Loading of support beam masonry	$W_{sb} = (h_{m, sb} \times b_{m, sb} \times \rho_{m, sb}) \times \gamma_G = 4.86 \text{ kN/m}$
Loading of torsion beam masonry	$W_{tb} = (h_{m, tb} \times b_{m, tb} \times \rho_{m, tb} + G_{k, add, tb}) \times \gamma_G + Q_{k, add, tb} \times \gamma_Q = 9.72 \text{ kN/m}$
Self weight of support beam	$W_{sw, sb} = A_{pl} \times \rho_{SEC3} \times g_{acc} \times \gamma_G = 0.36 \text{ kN/m}$
Self weight of torsion beam	$W_{sw, tb} = A_{tb} \times \rho_{SEC3} \times g_{acc} \times \gamma_G = 0.29 \text{ kN/m}$

Torsional loading SLS

Loading of support beam masonry	$W_{sb, ser} = h_{m, sb} \times b_{m, sb} \times \rho_{m, sb} = 3.60 \text{ kN/m}$
Loading of torsion beam masonry	$W_{tb, ser} = h_{m, tb} \times b_{m, tb} \times \rho_{m, tb} + G_{k, add, tb} + Q_{k, add, tb} = 7.00 \text{ kN/m}$
Self weight of support beam	$W_{sw, sb, ser} = A_{pl} \times \rho_{SEC3} \times g_{acc} = 0.27 \text{ kN/m}$
Self weight of torsion beam	$W_{sw, tb, ser} = A_{tb} \times \rho_{SEC3} \times g_{acc} = 0.22 \text{ kN/m}$

Torsional effects

Applied torque (ULS)	$T_{d, w} = \text{abs}(W_{sb} \times e_{m, sb} + W_{tb} \times e_{m, tb} + W_{sw, sb} \times e_{b, sb} + W_{sw, tb} \times e_{b, tb}) = 1.25 \text{ kNm/m}$
Total torque (ULS)	$T_d = T_{d, w} \times L = 2.88 \text{ kNm}$
Applied torque (SLS)	$T_{d, w, ser} = \text{abs}(W_{sb, ser} \times e_{m, sb} + W_{tb, ser} \times e_{m, tb} + W_{sw, sb, ser} \times e_{b, sb} + W_{sw, tb, ser} \times e_{b, tb}) = 0.93 \text{ kNm/m}$
Total torque (SLS)	$T_{d, ser} = T_{d, w, ser} \times L = 2.13 \text{ kNm}$

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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 1.0.06

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 150x100x6.0 (Tata Steel (old))
Steel grade - EN 10025-2:2004 S275
Nominal thickness of element $t_{nom} = t = 6 \text{ mm}$
Nominal yield strength $f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength $f_u = 410 \text{ N/mm}^2$
Modulus of elasticity $E = 210000 \text{ N/mm}^2$

Shear centre

Distance between flange shear centres $h_s = h - t = 144.0 \text{ mm}$
Shear centre (above bottom flange centroid) $e_{s,bf} = h_s / 2 = 72.0 \text{ mm}$

Torsional section modulus

Perimeter length $p = 2 \times ((h - t) + (b - t)) - 2 \times 1.25 \times t \times (4 - \pi) = 463 \text{ mm}$
Area enclosed by mean perimeter $A_p = (h - t) \times (b - t) - (1.25 \times t)^2 \times (4 - \pi) = 13488 \text{ mm}^2$
Torsional section modulus $W_t = I_t / (t + 2 \times A_p / p) = 147 \text{ cm}^3$

Analysis results

Design bending moment - major axis $M_{y,Ed} = 10.1 \text{ kNm}$
Design shear force - major axis $V_{y,Ed} = 17.5 \text{ kN}$

Classification

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

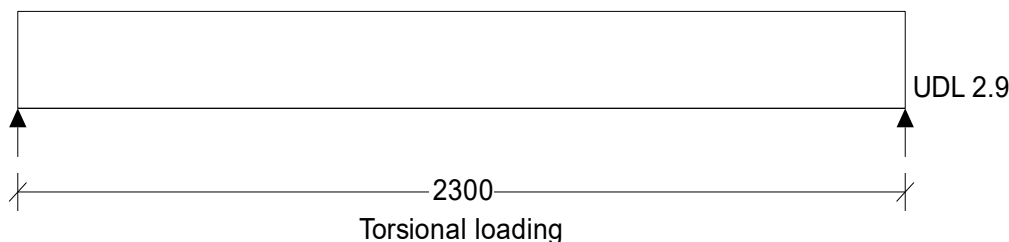
Width of section $c = h - 3 \times t = 132 \text{ mm}$
 $c / t = 22 = 23.8 \times \epsilon \leq 72 \times \epsilon$ Class 1

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

Width of section $c = b - 3 \times t = 82 \text{ mm}$
 $c / t = 13.7 = 14.8 \times \epsilon \leq 33 \times \epsilon$ Class 1

Section is class 1

Torsional loads



Load No.	Load type	Load (kNm)	Distance along beam (mm)
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Load No.	Load type	Load (kNm)	Distance along beam (mm)
1	UDL	2.9	-

Design torque at LHS support $T_A = T_{d,1} / 2 = \mathbf{1.44 \text{ kNm}}$
 Design torque at RHS support $T_B = T_{d,1} / 2 = \mathbf{1.44 \text{ kNm}}$
 Average torque over haf the beam $T_{av,t,Ed} = - T_{d,1} / 4 = \mathbf{-0.72 \text{ kNm}}$
 Maximum St Venant torsion design moment $T_{t,Ed} = \max(\text{abs}(T_A), \text{abs}(T_B)) = \mathbf{1.44 \text{ kNm}}$
 Rotation at mid-span $\phi = \text{abs}(T_{av,t,Ed}) \times L / (2 \times G_{SEC3} \times I_t) = \mathbf{0.00108}$
 Additional minor axis moment $M_{z,add,Ed} = \phi \times M_{y,Ed} = \mathbf{0.01 \text{ kNm}}$

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t = \mathbf{138 \text{ mm}}$
 $\eta = \mathbf{1.000}$
 $h_w / t = 23 = 24.9 \times \varepsilon / \eta < 72 \times \varepsilon / \eta$
Shear buckling resistance can be ignored

Design shear force $V_{y,Ed} = \mathbf{17.52 \text{ kN}}$
 Shear area - cl 6.2.6(3) $A_v = A \times h / (b + h) = \mathbf{1690 \text{ mm}^2}$
 Design shear resistance - cl 6.2.6(2) $V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{268.4 \text{ kN}}$
 Shear stress due to St Venant torsion $\tau_{t,Ed} = T_{t,Ed} / W_t = \mathbf{9.76 \text{ N/mm}^2}$
 Reduced shear resistance due to torsion - eq 6.26 $V_{c,y,Rd} = V_{pl,T,y,Rd} = (1 - \tau_{t,Ed} / ((f_y / \sqrt{3}) / \gamma_{M0})) \times V_{pl,y,Rd} = \mathbf{251.9 \text{ kN}}$
 $V_{y,Ed} / V_{pl,T,y,Rd} = \mathbf{0.07}$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment $M_{y,Ed} = \mathbf{10.1 \text{ kNm}}$
 Design bending resistance moment - eq 6.13 $M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{38.7 \text{ kNm}}$
 $M_{y,Ed} / M_{pl,y,Rd} = \mathbf{0.261}$
PASS - Design bending resistance moment exceeds design bending moment

Check bending moment - Section 6.2.5

Design bending moment $M_{z,Ed,total} = M_{z,Ed} + M_{z,add,Ed} = \mathbf{0 \text{ kNm}}$
 Design bending resistance moment - eq 6.13 $M_{c,z,Rd} = M_{pl,z,Rd} = W_{pl,z} \times f_y / \gamma_{M0} = \mathbf{29.1 \text{ kNm}}$
 $M_{z,Ed,total} / M_{pl,z,Rd} = \mathbf{0}$
PASS - Design bending resistance moment exceeds design bending moment

Torsional resistance

Design resistance to St. Venant torsion $T_{Rd} = C \times f_y / (\sqrt{3} \times \gamma_{M0}) = \mathbf{23.38 \text{ kNm}}$
 $T_{t,Ed} / T_{Rd} = \mathbf{0.061}$
PASS - Design torsional resistance exceeds applied St Venants torsion

Plastic verification - cl.6.2.9.1(6)

$\alpha_{bi} = \mathbf{1.66}$
 $\beta_{bi} = \mathbf{1.66}$
 $[M_{y,Ed} / M_{pl,y,Rd}]^{\alpha_{bi}} + [M_{z,Ed,total} / M_{pl,z,Rd}]^{\beta_{bi}} = \mathbf{0.107}$
PASS - Plastic interaction criterion is less than 1.0

Serviceability limit checks

Rotation limit $\phi_{ser,lim} = \mathbf{3.00 \text{ deg}}$
 Rotation of torsion beam $\phi_{ser} = M_{z,add,Ed} \times 180 / (M_{y,Ed} \times \gamma_G \times \pi) = \mathbf{0.05 \text{ deg}}$
PASS - Rotation limit exceeds rotation in torsion beam

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Vertical deflection limit

$$\delta_{v,lim} = 10.0 \text{ mm}$$

SLS loading on beam

$$f_{d,ser} = W_{sb,ser} + W_{tb,ser} + W_{sw,sb,ser} + W_{sw,tb,ser} = 11.09 \text{ kN/m}$$

Vertical deflection of torsion beam

$$\delta_v = 5 \times f_{d,ser} \times L^4 / (384 \times E_{sb} \times I_{ytb}) = 2.2 \text{ mm}$$

PASS - Vertical deflection limit exceeds vertical deflection in torsion beam

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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	29.7 kN	38.2 kN	N/A	N/A	0.778	Pass

Masonry panel details

Panel length	$L = 1300$ mm
Panel height	$h = 2000$ mm
Thickness of load bearing leaf	$t = 165$ mm
Effective height	$h_{ef} = 2000$ mm
Effective thickness	$t_{ef} = 165$ mm

Masonry material details

Unit type	Natural stone - Group 1
Compressive strength of masonry unit	$f_c = 7.3$ N/mm ²
Height of unit	$h_u = 215$ mm
Width of unit	$w_u = 215$ mm
Conditioning factor	$k = 1.0$
- Conditioning to the air dry condition in accordance with cl.7.3.2	
Shape factor - Table A.1	$d_{sf} = 1.16$
Mean compressive strength of masonry unit	$f_b = f_c \times k \times d_{sf} = 8.47$ N/mm ²
Specific weight of units	$\gamma = 20$ 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	$K = 0.45$
Characteristic compressive strength - eq. 3.1	$f_k = K \times f_b^{0.7} \times f_m^{0.3} = 3.04$ N/mm ²
Short term secant modulus of elasticity factor	$K_E = 1000$
Modulus of elasticity - cl.3.7.2	$E_w = K_E \times f_k = 3044$ 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	$\gamma_M = 3.00$
Cross-sectional area of wall	$A = L \times t = 0.21$ m ²
Design compressive strength of masonry	$f_d = f_k / \gamma_M = 1.01$ N/mm ²

Partial safety factors for design loads

Partial safety factor for permanent load	$\gamma_{FG} = 1.35$
Partial safety factor for variable load	$\gamma_{FQ} = 1.50$

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

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Eccentricity of permanent UDL load

$e_{gu} = 0 \text{ mm}$

Eccentricity of variable UDL load

$e_{qu} = 0 \text{ mm}$

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit

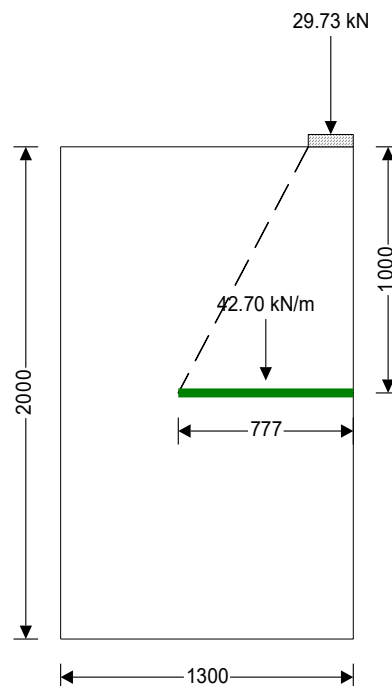
$\lambda_{lim} = 27$

Slenderness ratio

$\lambda = h_{ef} / t_{ef} = 12.1$

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load

$G_{kc1} = 14.80 \text{ kN}$

Variable concentrated load

$Q_{kc1} = 6.50 \text{ kN}$

Eccentricity of concentrated load

$e_{c1} = 0 \text{ mm}$

Length of concentrated load

$L_{c1} = 200 \text{ mm}$

Width of concentrated load

$w_{c1} = 152 \text{ mm}$

Height of concentrated load

$h_{c1} = 2000 \text{ mm}$

Distance of load to right vertical edge

$r_{11} = 0 \text{ mm}$

Distance of load to nearest vertical edge

$a_{11} = 0 \text{ mm}$

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check

$e_{c1} \leq t / 4$

PASS - Eccentricity of load is less than $t/4$

Area of bearing

$A_{b1} = L_{c1} \times w_{c1} = 30400 \text{ mm}^2$

Effective length of bearing at mid-height

$l_{efm1} = L_{c1} + h_{c1} / 2 \times \tan(30) + r_{11} = 777 \text{ mm}$

Effective bearing area

$A_{ef1} = l_{efm1} \times t = 128263 \text{ mm}^2$

Bearing area ratio check

$A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = 0.24$

Initial enhancement factor

$\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = 1.24$

Maximum enhancement factor

$\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = 1.25$

Enhancement factor for concentrated loads

$\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1}) = 1.24$

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Design value of the concentrated load

$$N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ} = \mathbf{29.73 \text{ kN}}$$

Design value concentrated load resistance

$$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d = \mathbf{38.22 \text{ kN}}$$

PASS - Design resistance exceeds applied concentrated load

Walls subjected to mainly vertical loading - Section 6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

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

Eccentricity of variable UDL at mid-height below concentrated load

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

Eccentricity of concentrated load at mid-height

$$e_{mc1} = e_{c1} / 2 = \mathbf{0.0 \text{ mm}}$$

Initial eccentricity - cl.5.5.1.1(4)

$$e_{init} = h_{ef} / 450 = \mathbf{4.4 \text{ mm}}$$

Concentrated load at mid-height as UDL

$$N_{mc1} = N_{Edc1} / l_{efm1} = \mathbf{38.25 \text{ kN/m}}$$

Vertical load at mid-height

$$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = \mathbf{42.70 \text{ kN/m}}$$

Design moment at mid-height

$$M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} = \mathbf{0.00 \text{ kNm/m}}$$

Eccentricities due to loads - eq. 6.7

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = \mathbf{4.4 \text{ mm}}$$

Slenderness ratio limit for creep eccentricity

$$\lambda_c = \mathbf{27}$$

Eccentricity due to creep

$$e_{k1} = \mathbf{0.0 \text{ mm}}$$

Eccentricity at mid-height - eq. 6.6

$$e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \times t) = \mathbf{8.3 \text{ mm}}$$

From eq. G2

$$A_{11} = 1 - 2 \times e_{mk1} / t = \mathbf{0.90}$$

From eq. G3

$$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = \mathbf{0.48}$$

Capacity reduction factor - eq. G1

$$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) = \mathbf{0.80}$$

Design vertical resistance of panel - eq.6.2

$$N_{Rd1} = \Phi_{m1} \times t \times f_d = \mathbf{134.46 \text{ kN/m}}$$

PASS - Design value of vertical resistance exceeds applied vertical load

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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	10.9 kN	28.5 kN	N/A	N/A	0.382	Pass

Masonry panel details

Panel length	$L = 500$ mm
Panel height	$h = 2000$ mm
Thickness of load bearing leaf	$t = 100$ mm
Effective height	$h_{ef} = 2000$ 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	$\gamma = 18$ 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	$\gamma_M = 3.00$
Cross-sectional area of wall	$A = L \times t = 0.05$ m ² ($A < 0.1$ m ²)
Design compressive strength of masonry	$f_d = f_k / \gamma_M \times (0.7 + 3 \times A / 1 \text{ m}^2) = 1.62$ N/mm ²

Partial safety factors for design loads

Partial safety factor for permanent load	$\gamma_{FG} = 1.35$
Partial safety factor for variable load	$\gamma_{FQ} = 1.50$

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

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Eccentricity of permanent UDL load

$e_{gu} = 0 \text{ mm}$

Eccentricity of variable UDL load

$e_{qu} = 0 \text{ mm}$

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit

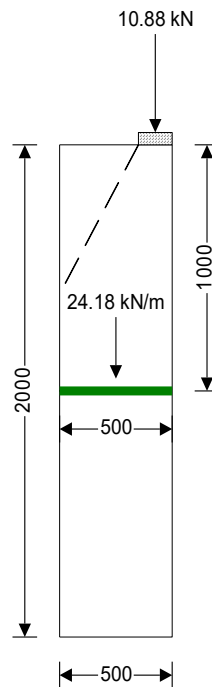
$\lambda_{lim} = 27$

Slenderness ratio

$\lambda = h_{ef} / t_{ef} = 20.0$

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load

$G_{kc1} = 4.50 \text{ kN}$

Variable concentrated load

$Q_{kc1} = 3.20 \text{ kN}$

Eccentricity of concentrated load

$e_{c1} = 0 \text{ mm}$

Length of concentrated load

$L_{c1} = 150 \text{ mm}$

Width of concentrated load

$w_{c1} = 100 \text{ mm}$

Height of concentrated load

$h_{c1} = 2000 \text{ mm}$

Distance of load to right vertical edge

$r_{11} = 0 \text{ mm}$

Distance of load to nearest vertical edge

$a_{11} = 0 \text{ mm}$

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check

$e_{c1} \leq t / 4$

PASS - Eccentricity of load is less than $t/4$

Area of bearing

$A_{b1} = L_{c1} \times w_{c1} = 15000 \text{ mm}^2$

Effective length of bearing at mid-height

$l_{efm1} = L = 500 \text{ mm}$

Effective bearing area

$A_{ef1} = l_{efm1} \times t = 50000 \text{ mm}^2$

Bearing area ratio check

$A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = 0.30$

Initial enhancement factor

$\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = 1.17$

Maximum enhancement factor

$\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = 1.25$

Enhancement factor for concentrated loads

$\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1}) = 1.17$

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Design value of the concentrated load

$$N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ} = \mathbf{10.88 \text{ kN}}$$

Design value concentrated load resistance

$$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d = \mathbf{28.48 \text{ kN}}$$

PASS - Design resistance exceeds applied concentrated load

Walls subjected to mainly vertical loading - Section 6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

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

Eccentricity of variable UDL at mid-height below concentrated load

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

Eccentricity of concentrated load at mid-height

$$e_{mc1} = e_{c1} / 2 = \mathbf{0.0 \text{ mm}}$$

Initial eccentricity - cl.5.5.1.1(4)

$$e_{init} = h_{ef} / 450 = \mathbf{4.4 \text{ mm}}$$

Concentrated load at mid-height as UDL

$$N_{mc1} = N_{Edc1} / l_{efm1} = \mathbf{21.75 \text{ kN/m}}$$

Vertical load at mid-height

$$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = \mathbf{24.18 \text{ kN/m}}$$

Design moment at mid-height

$$M_{Ed1} = g_k \times \gamma_{fG} \times e_{gmu1} + q_k \times \gamma_{fQ} \times e_{qmu1} + N_{mc1} \times e_{mc1} = \mathbf{0.00 \text{ kNm/m}}$$

Eccentricities due to loads - eq. 6.7

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = \mathbf{4.4 \text{ mm}}$$

Slenderness ratio limit for creep eccentricity

$$\lambda_c = \mathbf{27}$$

Eccentricity due to creep

$$e_{k1} = \mathbf{0.0 \text{ mm}}$$

Eccentricity at mid-height - eq. 6.6

$$e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \times t) = \mathbf{5.0 \text{ mm}}$$

From eq. G2

$$A_{11} = 1 - 2 \times e_{mk1} / t = \mathbf{0.90}$$

From eq. G3

$$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = \mathbf{0.85}$$

Capacity reduction factor - eq. G1

$$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) = \mathbf{0.63}$$

Design vertical resistance of panel - eq.6.2

$$N_{Rd1} = \Phi_{m1} \times t \times f_d = \mathbf{101.93 \text{ kN/m}}$$

PASS - Design value of vertical resistance exceeds applied vertical load

Project 13 Tyntaldwyn Road, Merthyr Tydfil, CF48 4NG				Job Ref. AUX609	
Section 5 Floor joists				Sheet no./rev. 05/ 1	
Calc. by MK	Date 30-10-2025	Chk'd by MK	Date 30-10-2025	App'd by	Date 30-10-2025

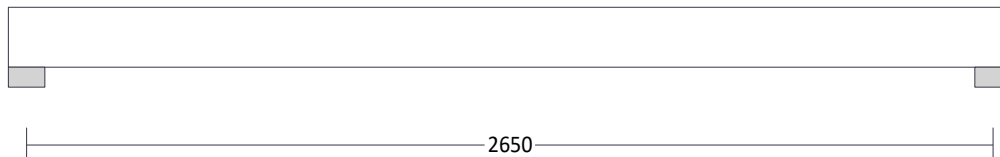
TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.09

Joist details

Description 50 x 150 C24 timber joists
Joist spacing $S_{Joist} = 450$ mm



Forces input on Joist

Vertical permanent load on joist $F_{G_Joist} = 0.50$ kN/m²
Vertical imposed load on joist $F_{Q_Joist} = 2.00$ kN/m²

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G_Joist} \times S_{Joist} = 0.23$ kN/m
Vertical imposed load on joist $p_Q = F_{Q_Joist} \times S_{Joist} = 0.90$ kN/m

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	1.7	0.3	0.203	PASS
Bending stress	N/mm ²	16.2	7.9	0.487	PASS
Shear stress	N/mm ²	2.7	0.7	0.247	PASS
Beam stability check				0.487	PASS
Deflection	mm	10.6	9.0	0.851	PASS

ANALYSIS

Tedds calculation version 1.0.38

Loading

Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.23 kN/m at 0 m to 2.65 m
Member	Imposed	UDL	GlobalZ	0.9 kN/m at 0 m to 2.65 m

Project 13 Tyntaldwyn Road, Merthyr Tydfil, CF48 4NG				Job Ref. AUX609	
Section 5 Floor joists				Sheet no./rev. 05/ 2	
Calc. by MK	Date 30-10-2025	Chk'd by MK	Date 30-10-2025	App'd by	Date 30-10-2025

Results

Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.48495	
2	0	0	-0.48495	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.3305	
2	0	0	-0.3305	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	4.5
1.00G + 1.00Q (Service)	0	3

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.65	1	0	-2.2	0
		2	0	-2.2	0

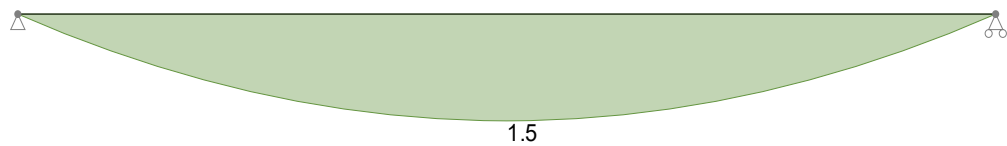
Project 13 Tyntaldwyn Road, Merthyr Tydfil, CF48 4NG				Job Ref. AUX609	
Section 5 Floor joists				Sheet no./rev. 05/ 3	
Calc. by MK	Date 30-10-2025	Chk'd by MK	Date 30-10-2025	App'd by	Date 30-10-2025

Load combination: 1.00G + 1.00Q (Service)

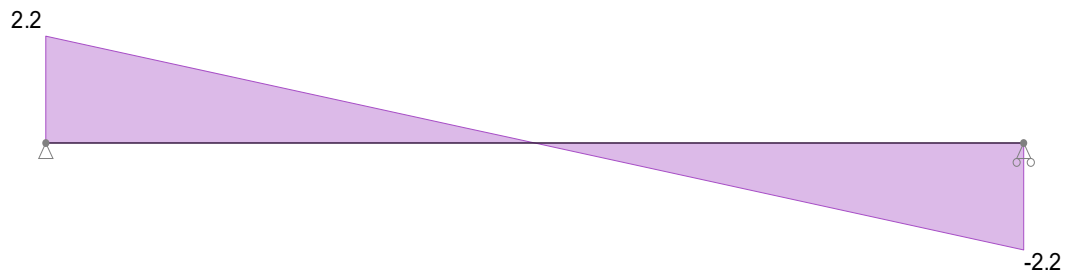
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.65	1	0	-1.5	0
		2	0	-1.5	0

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	2.2 (max abs)		0 (min)	
	1.325	0		1.5 (max)	
	2.65	-2.2		0 (min)	

Tedds calculation version 2.2.23

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 $\gamma_M = 1.300$

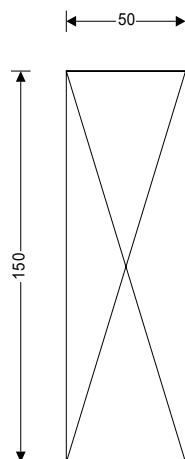
Member details

Load duration - cl.2.3.1.2 Medium-term
Service class - cl.2.3.1.3 2

Timber section details

Number of timber sections in member $N = 1$
Breadth of sections $b = 50$ mm
Depth of sections $h = 150$ mm
Timber strength class - EN 338:2016 Table 1 **C24**

Project 13 Tyntaldwyn Road, Merthyr Tydfil, CF48 4NG				Job Ref. AUX609	
Section 5 Floor joists				Sheet no./rev. 05/ 4	
Calc. by MK	Date 30-10-2025	Chk'd by MK	Date 30-10-2025	App'd by	Date 30-10-2025



50x150 timber section

Cross-sectional area, A , 7500 mm²

Section modulus, W_y , 187500 mm³

Section modulus, W_z , 62500 mm³

Second moment of area, I_y , 14062500 mm⁴

Second moment of area, I_z , 1562500 mm⁴

Radius of gyration, i_y , 43.3 mm

Radius of gyration, i_z , 14.4 mm

Timber strength class C24

Characteristic bending strength, $f_{m,k}$, 24 N/mm²

Characteristic shear strength, $f_{v,k}$, 4 N/mm²

Characteristic compression strength parallel to grain, $f_{c,0,k}$, 21 N/mm²

Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.5 N/mm²

Characteristic tension strength parallel to grain, $f_{t,0,k}$, 14.5 N/mm²

Mean modulus of elasticity, $E_{0,mean}$, 11000 N/mm²

Fifth percentile modulus of elasticity, $E_{0.05}$, 7400 N/mm²

Shear modulus of elasticity, G_{mean} , 690 N/mm²

Characteristic density, ρ_k , 350 kg/m³

Mean density, ρ_{mean} , 420 kg/m³

Span details

Bearing length

$L_b = 100$ mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.8$

Deformation factor - Table 3.2 $k_{def} = 0.8$

Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 0.7$

Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$

System strength factor - cl.6.6 $k_{sys} = 1.1$

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 2.237$ kN

Effective contact length $L_{b,ef} = L_b + \min(L_b, 30 \text{ mm}) = 130$ mm

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.344$ N/mm²

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.692$ N/mm²

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.203$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 2.237$ kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.668$ N/mm²

Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.708$ N/mm²

$\tau_{y,d} / f_{v,y,d} = 0.247$

PASS - Design shear strength exceeds design shear stress

Check design 1325 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 1.482$ kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 7.905$ N/mm²

Project 13 Tyntaldwyn Road, Merthyr Tydfil, CF48 4NG				Job Ref. AUX609	
Section 5 Floor joists				Sheet no./rev. 05/ 5	
Calc. by MK	Date 30-10-2025	Chk'd by MK	Date 30-10-2025	App'd by	Date 30-10-2025

Design bending strength

$$f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = \mathbf{16.246 \text{ N/mm}^2}$$

$$\sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.487}$$

PASS - Design bending strength exceeds design bending stress

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Lateral buckling factor - exp.6.34

$$k_{crit} = \mathbf{1.000}$$

Beam stability check - exp.6.33

$$\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}) = \mathbf{0.487}$$

PASS - Beam stability is acceptable

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 1325 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$$\delta_y = \mathbf{5 \text{ mm}}$$

Quasi-permanent variable load factor

$$\psi_2 = \mathbf{0.3}$$

Final deflection with creep

$$\delta_{y,Final} = \delta_y \times (1 + k_{def}) = \mathbf{9 \text{ mm}}$$

Allowable deflection

$$\delta_{y,Allowable} = L_{m1_s1} / 250 = \mathbf{10.6 \text{ mm}}$$

$$\delta_{y,Final} / \delta_{y,Allowable} = \mathbf{0.851}$$

PASS - Allowable deflection exceeds final deflection

STRUCTURAL CALCULATIONS

Using
MASTERSERIES POWERPAD

AUX609

RETAINING WALL A

MasterSeries Sample Output

3 Castle Street
Carrickfergus,
County Antrim, BT38 7BE

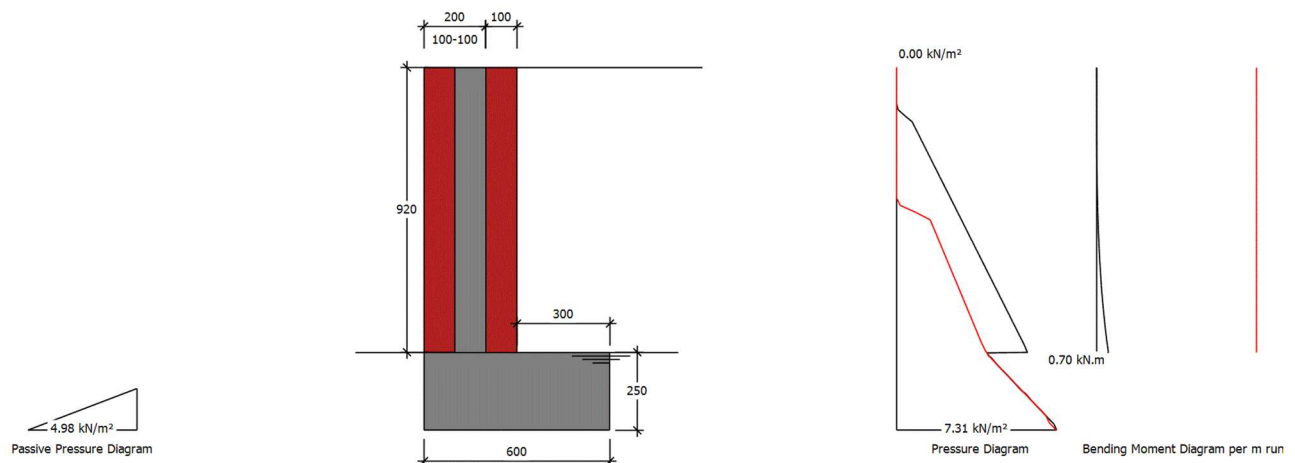
Tel: (028) 9036 5950

Job ref : Job Ref
Sheet : Sheet Ref / 2 -
Made By :
Date : 31 Oct 2025/ Version 2015.04
Checked :
Approved :

MASTERKEY : RETAINING WALL DESIGN TO BS 8002 : 1994, BS EN 1996-1-1:2005+A1:2012 AND BS 8110 : 1997

Basic RC Retaining Wall

Reinforced Masonry Cavity Filled Retaining Wall with Un-Reinforced Concrete Base



Summary of Design Data

EC6 National Annex	UK
Notes	All dimensions are in mm and all forces are per metre run
Material Densities (kN/m³)	Dry Soil 18.00, Saturated Soil 20.80, Submerged Soil 10.80
	Concrete 24.00, Masonry 20.00
Concrete grade	fcu 30 N/mm², Permissible tensile stress 0.250 N/mm²
Concrete covers (mm)	Cavity inner cover 35 mm, Cavity outer cover 55 mm
Reinforcement design	fy 500 N/mm², fyb 500 N/mm² designed to BS 8110: 1997
Surcharge and Water Table	Surcharge 5.00 kN/m², Water table level 0 mm
Unplanned excavation depth	Front of wall 117 mm
† The Engineer must satisfy him/herself to the reinforcement detailing requirements of the relevant codes of practice	

Soil Properties

Bearing pressure	Permissible service pressure @ front 100.00 kN/m², @ back 100.00 kN/m²
Back Soil Friction and Cohesion	$\phi = \text{Atn}(\tan(30)/1.2) = 25.69^\circ$, $c = 3/1.5 = 2 \text{ kN/m}^2$
Base Friction and Cohesion	$\delta = \text{Atn}(0.75 \times \tan(\text{Atn}(\tan(30)/1.2))) = 19.84^\circ$, $c = 0.75 \times 3/1.5 = 1.5 \text{ kN/m}^2$
Front Soil Friction and Cohesion	$\phi = \text{Atn}(\tan(30)/1.2) = 25.69^\circ$

Loading Cases

G_{Soil} - Soil Self Weight, G_{Wall} - Wall & Base Self Weight, $F_{V\text{Heel}}$ - Vertical Loads over Heel,	
P_a - Active Earth Pressure, $P_{\text{surcharge}}$ - Earth pressure from surcharge, P_p - Passive Earth Pressure	
Case 1: Geotechnical Design	1.00 G_{Soil} +1.00 G_{Wall} +1.00 $F_{V\text{Heel}}$ +1.00 P_a +1.00 $P_{\text{surcharge}}$ +1.00 P_p
Case 2: Structural Ultimate Design	1.40 G_{Soil} +1.40 G_{Wall} +1.60 $F_{V\text{Heel}}$ +1.00 P_a +1.00 $P_{\text{surcharge}}$ +1.00 P_p

Geotechnical Design

Wall Stability - Virtual Back Pressure

Case 1 Overturning/Stabilising	0.715/4.819	0.148	OK
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Wall Sliding - Virtual Back Pressure

$F_x/(R_{X\text{Friction}} + R_{X\text{Passive}})$	2.678/(6.525+0.334)	0.390	OK
--	---------------------	-------	----

Soil Pressure

Virtual Back (No uplift)	Max(35.526/100, 16.434/100) kN/m²	0.355	OK
Wall Back (No uplift)	Max(48.193/100, 3.767/100) kN/m²	0.482	OK

MasterSeries Sample Output

3 Castle Street
Carrickfergus,
County Antrim, BT38 7BE

Tel: (028) 9036 5950

Job ref : **Job Ref**
Sheet : **Sheet Ref / 3 -**
Made By :
Date : **31 Oct 2025/ Version 2015.04**
Checked :
Approved :

Structural Design**Outer Leaf Details**

Partial Safety Factor (γ_{mc}/γ_{mf})	Units Category II, Execution Control Class 2	3/2.7	Table NA.1
Material	Clay bricks with water absorption less than 7%		
Units and Mortar Strength	5 N/mm ² , Mortar designation M12/(i)		
Compressive Strength (f_k)	Group 1, $\gamma=20$ kN/m ³	2.5 N/mm ²	Table NA.4

Inner Leaf Details

Partial Safety Factor (γ_{mc}/γ_{mf})	Units Category II, Execution Control Class 2	3/2.7	Table NA.1
Material	Concrete blocks, $\gamma=20$ kN/m ³		
Units and Mortar Strength	7.3 N/mm ² , Mortar designation M12/(i)		
Blocks Ratio	Unit height=215, Least horizontal dimensions=102.5	2.10	
Compressive Strength (f_k)	Group 1	5.48 N/mm ²	Table NA.6

Wall Design (Inner Steel)

Critical Section	Critical @ 0 mm from base, Case 2		
Steel Provided (Cover)	Dist. H10@200 (30 mm in cavity)		
	Main H10@200 (35 mm in cavity)	393 mm ²	OK
Leverarm $z=fn(d,b,A_s,f_y,f_k)$	160 mm, 1000 mm, 341 mm ² , 500 N/mm ² , 2.5 N/mm ²	80 mm	
$M_r=fn(above,\gamma_{mf},\gamma_{ms})$	2.70, 1.15	9.5 kN.m	
Moment Capacity Check (M/M _r)	M 0.7 kN.m, M _r 9.5 kN.m	0.073	OK
Shear Capacity Check	F 2.5 kN, f_{vk} 0.800 N/mm ² , F _{vr} 51.2 kN	0.05	OK

Base Top Face Tension

Maximum Tensile Stress	M 1.253, h 250, f _{cu} 30, Permissible 0.25 N/mm ²	0.12 N/mm ²	OK
Shear Capacity Check	F 3.9 kN, v _c 0.350 N/mm ² , F _{vr} 43.8 kN	0.09	OK

Base Bottom Face Tension

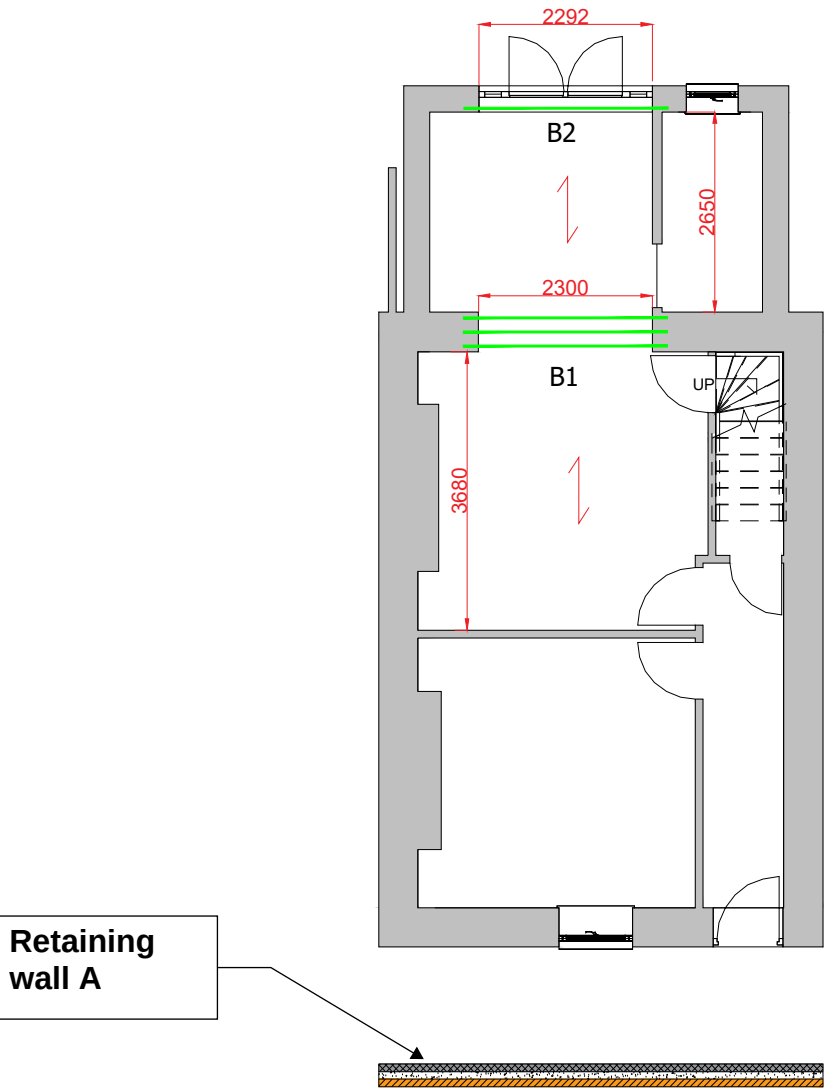
Maximum Tensile Stress	M 0.196, h 250, f _{cu} 30, Permissible 0.25 N/mm ²	0.02 N/mm ²	OK
Shear Capacity Check	F 2.6 kN, v _c 0.350 N/mm ² , F _{vr} 43.8 kN	0.06	OK

Job ref	: Job Ref
Sheet	: Sheet Ref / 4 -
Made By	:
Date	: 31 Oct 2025/ Version 2015.04
Checked	:
Approved	:

Tel: (028) 9036 5950

Note : Contractor must take his own measurements prior to ordering any structural material.

- NOTES:
1. ALL DIMENSIONS IN mm UNLESS NOTED OTHERWISE.
 2. DO NOT SCALE FROM THIS DRAWING.
 3. ALL DIMENSIONS TO BE VERIFIED ON SITE PRIOR TO COMMENCING WORKS

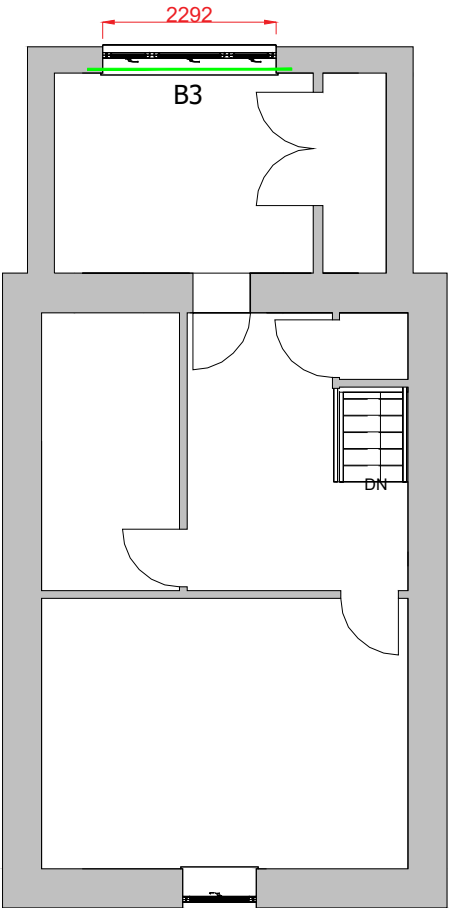


GROUND FLOOR PLAN SHOWS
STRUCTURE ABOVE

B1 = 3 x 152x152x23 UC S275
(Min end bearing - 200mm)

B2 = 150x100x6 RHS S275 with 350x10
S275 plate welded to bottom flange
(Min end bearing - 200mm)

50x150 C24 @450 C/C
Floor joists



B3 = 150x100x6 RHS S275 with 350x10
S275 plate welded to bottom flange
(Min end bearing - 200mm)

FIRST FLOOR PLAN SHOWS
STRUCTURE ABOVE



Client
Angharad Davies

Project
13 Tyntaldwyn Road, Merthyr Tydfil,
CF48 4NG

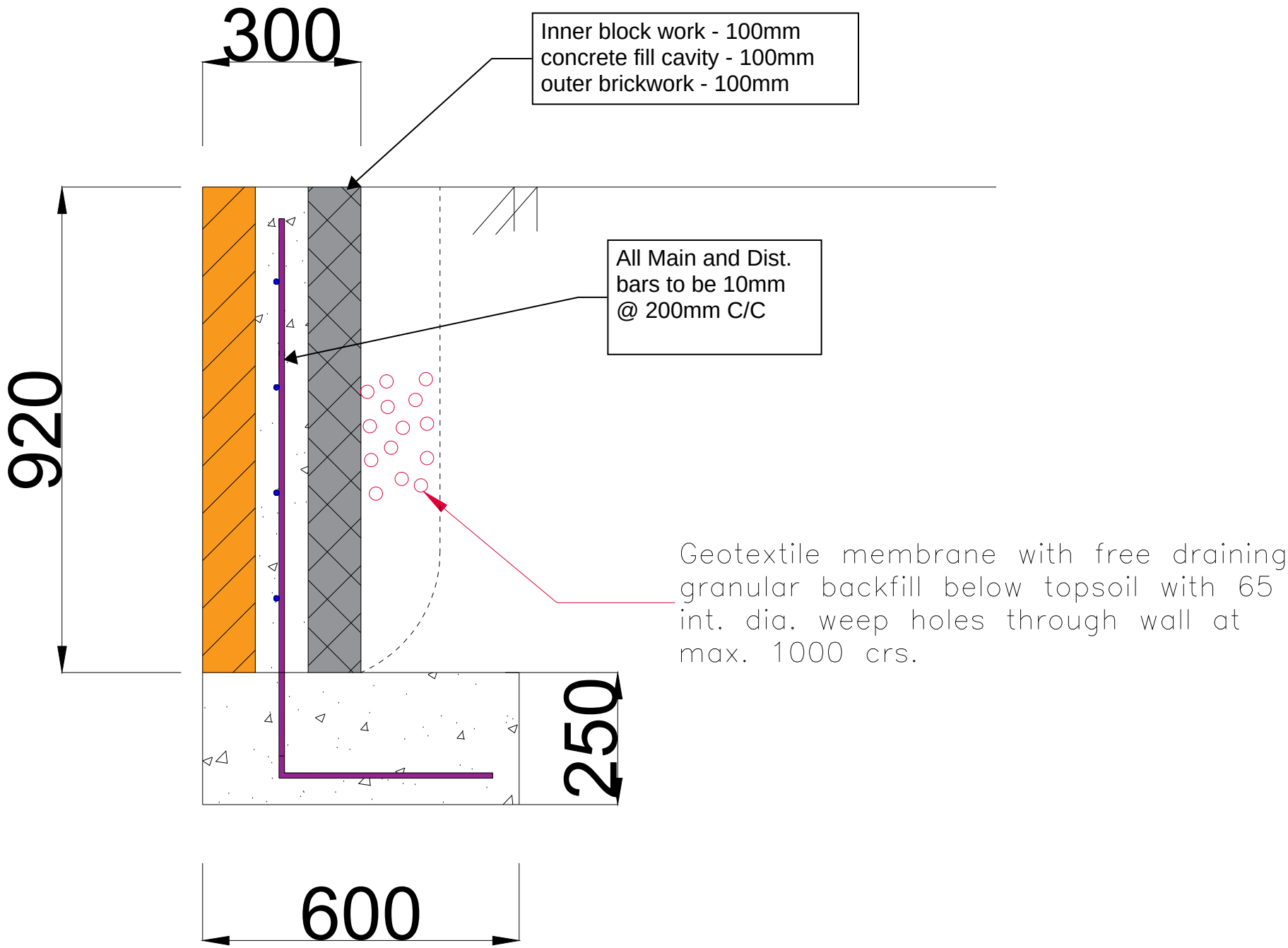
Drawing Title
Ground floor plan shows structure
above

Drawing Status
For Construction

Drawn By: MK	Date: 30/10/2025	Checked By:	Date:
Original Size A3	Scale NTS	Issue Date	
Project Number AUX- 609	Drawing Number 101	Revision C01	

Note : Contractor must take his own measurements prior to ordering any structural material.

- NOTES:
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 - 3. ALL DIMENSIONS TO BE VERIFIED ON SITE PRIOR TO COMMENCING WORKS



WALL A
Retaining wall section



Client
Angharad Davies

Project
13 Tyntaldwyn Road, Merthyr Tydfil,
CF48 4NG

Drawing Title
Retaining wall reinforcement

Drawing Status
For Construction

Drawn By: MK	Date: 30/10/2025	Checked By:	Date:
Original Size A3	Scale NTS	Issue Date	
Project Number AUX- 609	Drawing Number 102	Revision C01	

Installing Temporary Supports

Acrow Props and Strongboys:

Above the opening, remove small sections of plaster to reveal brick/block joints.

Install strongboys into the bed joints above the proposed beam position.

Place Acrow props underneath the strongboys at regular intervals (usually every 900–1200mm depending on wall and span).

Ensure props are plumb and on solid base plates or timber spreaders.

Double-Sided Support (if required):

For cavity or thicker walls, support both skins using back-to-back strongboys or needle beams with Acrows on both sides.

On installation of the steel beams below the retained section of wall and floor above, the conventional approach would be to shim and pack the gap between the top of the beam and the bottom of the retained section of wall. If the wall was no longer supporting the floor joists, any timber members would normally be shimmed by hardwood wedges. The shimming and packing should be installed in a sequential manner and well driven in to induce the downward deflection in the steel supporting beam to take up the load from above. In this way, when the propping was released from the structure above, the further downward deflection of the beam to take up the load being transferred to it would be minimal.

Note : Contractor must take his own measurements prior to ordering any structural material.

NOTES:

- 1. ALL DIMENSIONS IN mm UNLESS NOTED OTHERWISE.
- 2. DO NOT SCALE FROM THIS DRAWING.
- 3. ALL DIMENSIONS TO BE VERIFIED ON SITE PRIOR TO COMMENCING WORKS

Client			
Angharad Davies			
Project			
13 Tyntaldwyn Road, Merthyr Tydfil, CF48 4NG			
Drawing Title			
Notes			
Drawing Status			
For Construction			
Drawn By:	Date:	Checked By:	Date:
MK	30/10/2025		
Original Size	Scale	Issue Date	
A3	NTS		
Project Number	Drawing Number	Revision	
AUX- 609	103	C01	