

JUST BEAMS

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Project:65

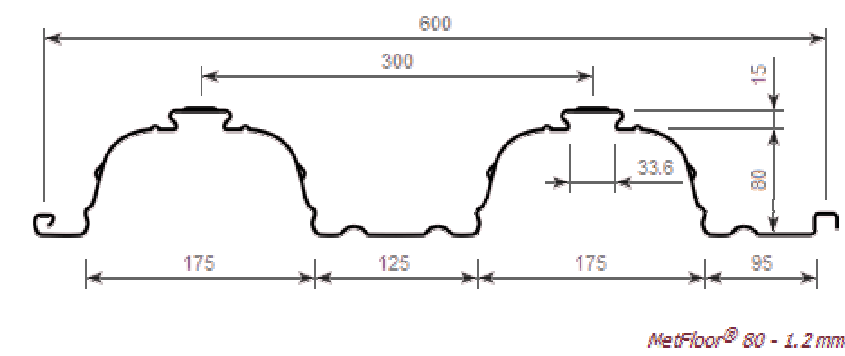
MAES Y COED GUEST HOUSE, MERTHYR
TYDFIL, CF47 8UY

STEEL BEAM DESIGN

- **Profile Details**

Profile Type	MetFloor® 80	Nominal Sheet Thickness	1.20 mm
Profile Yield	350 N/mm²	Effective Sheet Thickness	1.14 mm
Permanent Shuttering ⁹	Yes		

- **Cross Section Details**

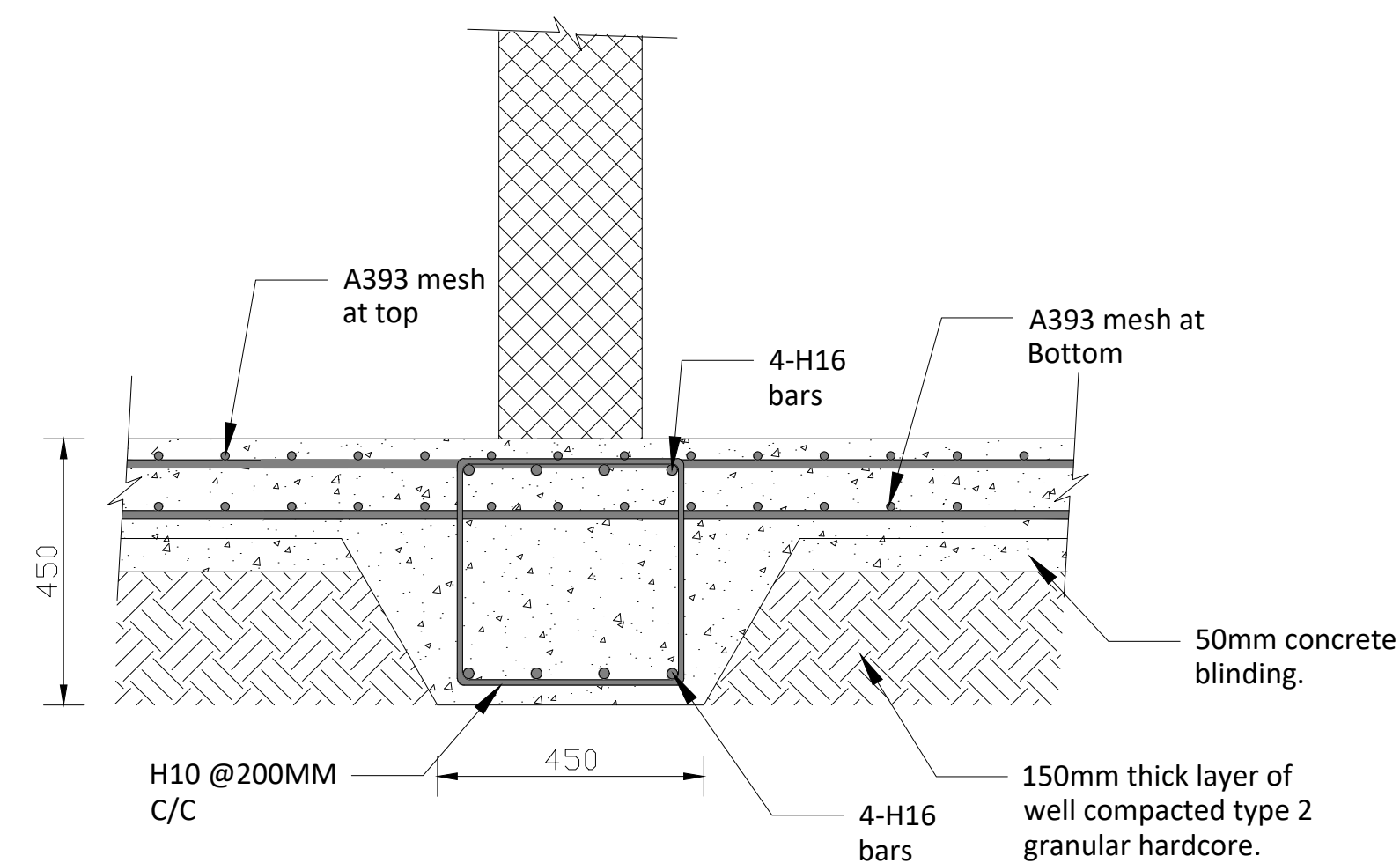
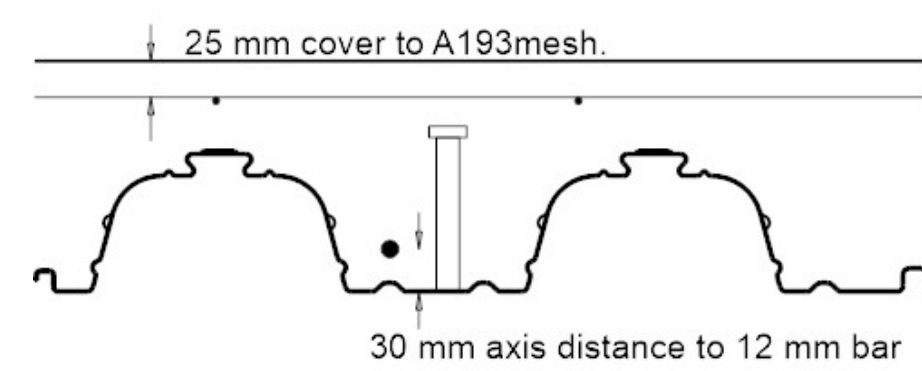


- **Slab Details**

Concrete Grade	C35/45	Slab Depth	160 mm
Concrete Type	Normal Weight	Modular Ratio	10
Wet Density	2550 kg/m ³	Dry Density	2450 kg/m ³
Mesh Type	A193	Mesh Cover	25
Mesh Layer	Single	Mesh Yield Stress	500 N/mm ²
Bar Diameter	12 mm	Bar Yield Stress	500 N/mm ²
		Axis Distance	30 mm
Account for End Anchorage	No	Number of Studs per Rib	N/A
Stud Diameter	N/A		
Screed Depth	50 mm	Screed Density	2400 kg/m ³

- **Slab Drawing**

MetFloor® 80 - 1.2 mm.



Internal Beam Detail

References & Sizes

LEGEND

		DENOTES STEEL BEAM
P1		DENOTE 215Wx440Lx215 Deep Padstone
P2		DENOTES 100Wx440Lx215 Deep Padstone
		DENOTES Metal Deck (See Section Details)

References & Sizes

Ref	Size & Description
B1	203x203x46 UC Steel Beam

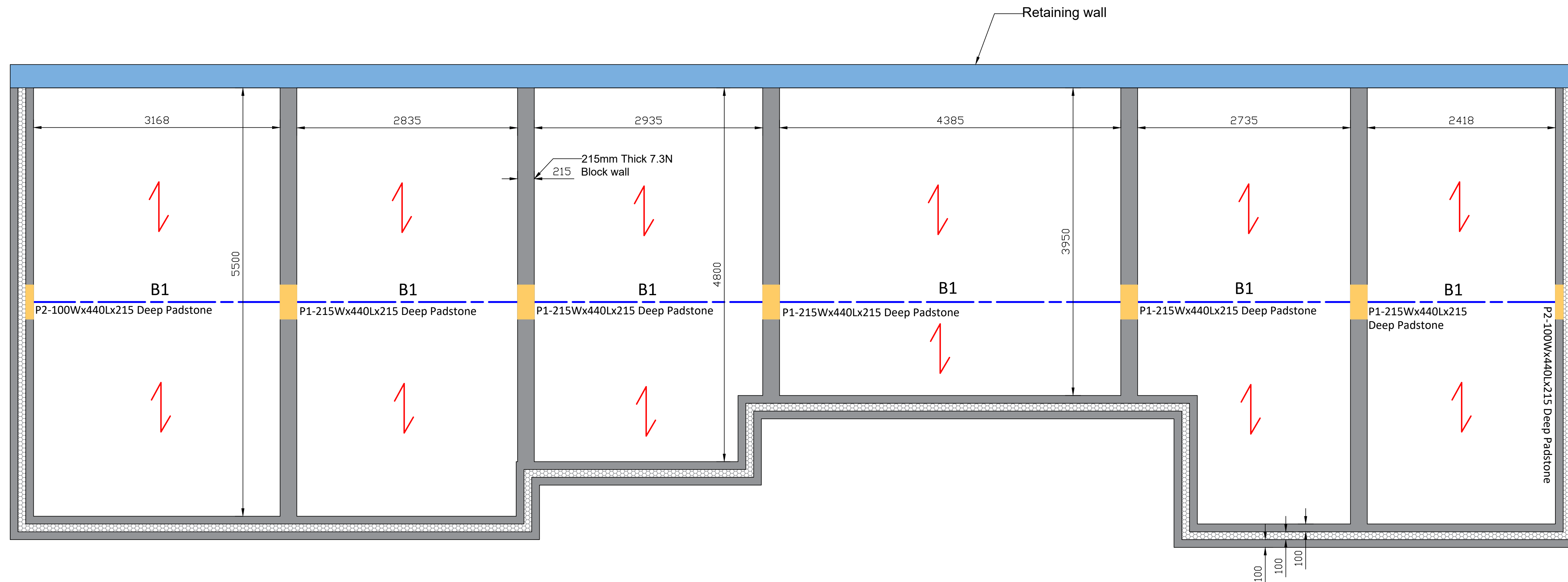
Note:

- 1- All internal walls are 215mm thick 7.3N block wall.
- 2- External walls are 300mm brick and block cavity walls.

DO NOT SCALE FROM THIS DRAWING. ALL
DIMENSIONS TO BE CONFIRMED ON SITE.

▼ NOTES

1. Drawings shall be referred to for structural work only unless otherwise stated and read in conjunction with Architect's, Services Engineers and Sub-Contractor's drawings.
2. The Contractor shall check all levels and dimensions, any errors or omissions are to be reported at once to the Engineer.



GROUND FLOOR PLAN SHOWING STRUCTURE ABOVE

Scale:1:30

-		ZR		ZR	ZR
REV	DATE	DRN	DESCRIPTION	CHK'D	APP'D
 REVISIONS					

REVISIONS

Status



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Job

**MAES Y COED GUEST HOUSE,
MERTHYR TYDFIL, CF47 8UY**

Title

**GROUND FLOOR PLAN
SHOWING STRUCTURE ABOVE**

Scale	Sheet Size	Date
1:30	A1	Nov 24

Drawing No.

001

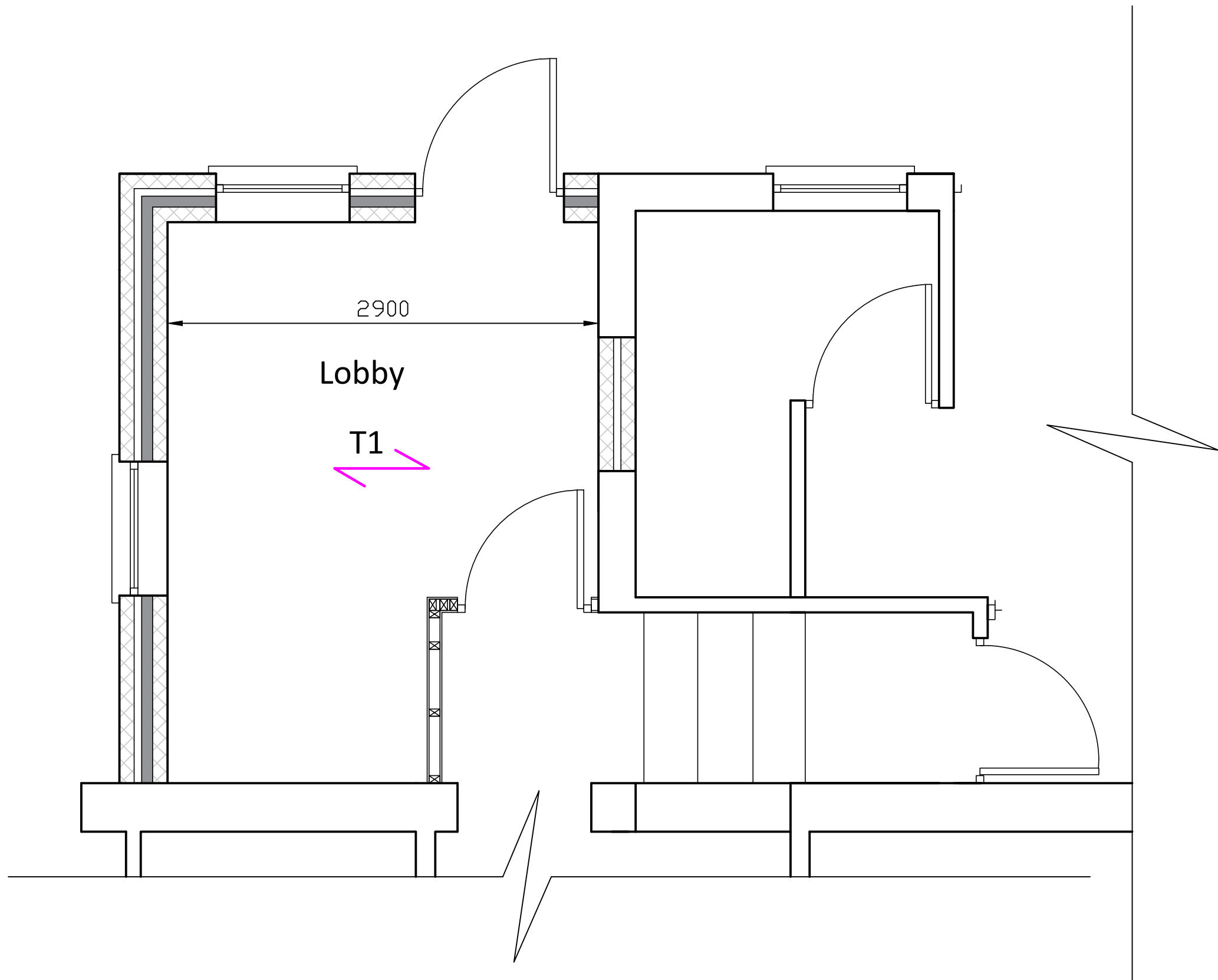
DO NOT SCALE FROM THIS DRAWING. ALL DIMENSIONS TO BE CONFIRMED ON SITE.

NOTES

- Drawings shall be referred to for structural work only unless otherwise stated and read in conjunction with Architect's, Services Engineers and Sub-Contractor's drawings.
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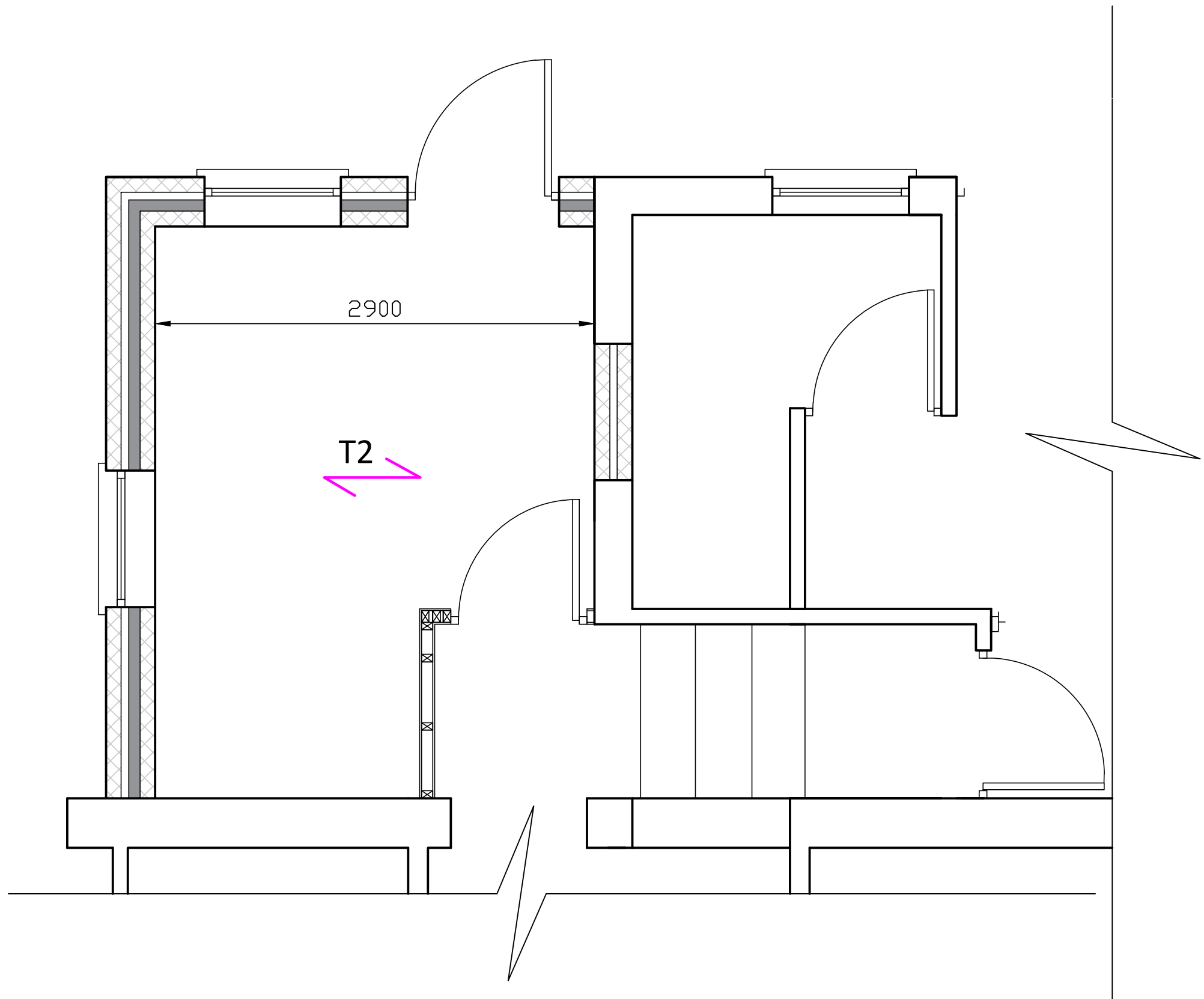
References & Sizes	
LEGEND	
	DENOTES TIMBER JOISTS

References & Sizes	
Ref	Size & Description
T1	47x200 C24 Timber Joists @450mm c/c
T2	47x170 C24 Timber Joists @450mm c/c



GROUND FLOOR PLAN SHOWING STRUCTURE ABOVE

Scale:1:30



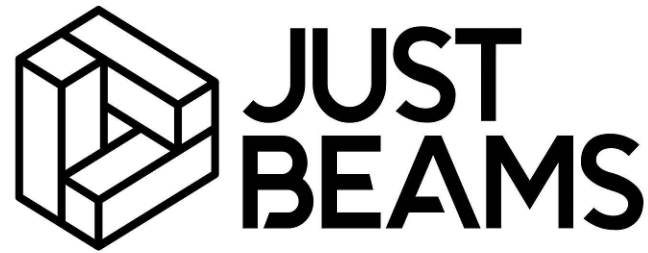
FIRST FLOOR PLAN SHOWING STRUCTURE ABOVE

Scale:1:30

REV	DATE	DRN	DESCRIPTION	CHK'D	APP'D
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Job

**MAES Y COED GUEST HOUSE,
MERTHYR TYDFIL, CF47 8UY**

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**GROUND FLOOR PLAN
SHOWING STRUCTURE ABOVE**

Scale	Sheet Size	Date
1:30	A1	Nov 24

Drawing No.

002

Project MAES Y COED GUEST HOUSE, MERTHYR TYDFIL, CF47 8UY				Status BUILDERS REVIEW		
Date By Checked	06-11-2024 Z.RAHMAN -	Job no. 65	Section -	Sheet no. 1	Rev R0	
Rev R0	Date 06-11-2024	Details STEEL BEAM DESIGN			Tel Fax	+44(0) 7949813086 -

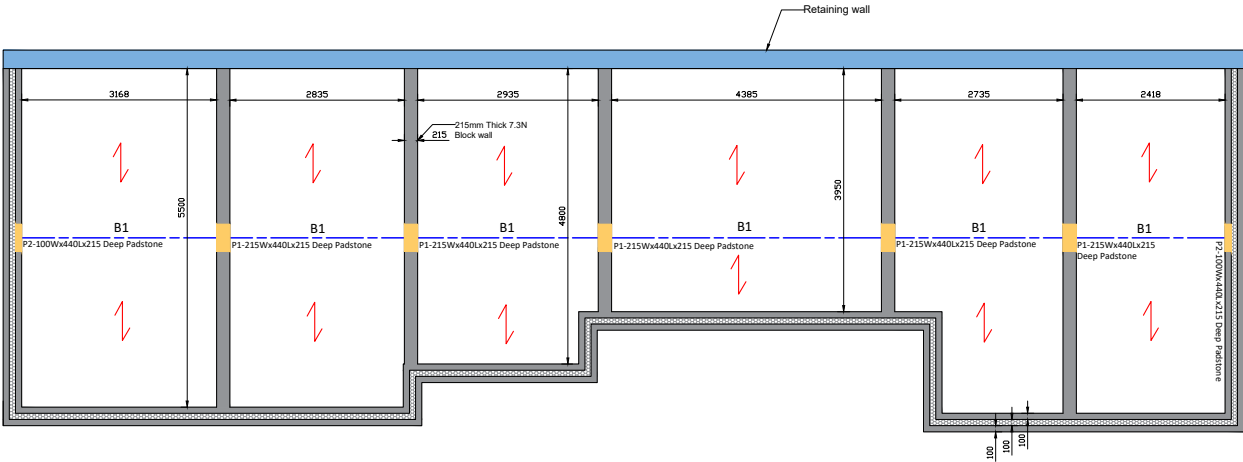
Part

INTRODUCTION

Ref:

INTRODUCTION

CLIENT WOULD LIKE TO HAVE CALCULATIONS FOR A NEW BUILDING.



GENERAL SPECIFICATION

- ALL UNSPECIFIED LEVELS AND SETTING OUT SHALL BE AS PER THE ARCHITECTURAL DRAWINGS AND CONFIRMED BY THE CONTRACTOR.

- THE CONTRACTOR IS TO DESIGN, SUPPLY, INSTALL, AND MAINTAIN ALL NECESSARY TEMPORARY PROPPING TO ENSURE VERTICAL AND LATERAL STABILITY TO ALL PARTS OF THE NEW AND EXISTING STRUCTURE.

- UNLESS NOTED OTHERWISE, IT HAS BEEN ASSUMED THAT THERE ARE SUITABLE FOUNDATIONS BENEATH ALL EXISTING LOAD-BEARING WALLS.

- THE LENGTHS OF ALL STRUCTURAL ELEMENTS (E.G. BEAMS, COLUMNS, RAFTERS ETC) MUST BE MEASURED BY THE CONTRACTOR TO SUIT ON-SITE BEFORE PLACING ANY ORDERS ON MATERIALS

- DO NOT SCALE FROM DRAWINGS OR TAKE DIMENSIONS FROM THE CALCULATIONS AND ANY DISCREPANCIES MUST BE REPORTED IMMEDIATELY TO THE STRUCTURAL ENGINEER.

- ALL WORKS MUST BE UNDERTAKEN BY A SUITABLY COMPETENT AND EXPERIENCED CONTRACTOR

Project MAES Y COED GUEST HOUSE, MERTHYR TYDFIL, CF47 8UY				Status BUILDERS REVIEW																																																									
Date By Checked	06-11-2024 Z.RAHMAN -	Job no. 65	Section -	Sheet no. 2	Rev R0																																																								
Rev R0	Date 06-11-2024	Details STEEL BEAM DESIGN			Tel Fax +44(0) 7949813086 -																																																								
Part																																																													
LOADINGS																																																													
Ref:		LOADINGS (SOURCE: NA TO BS EN 1991-1-1:2002) Dead Loadings Internal Timber Floor Build - Timber boards+joist = - Ceiling and services = Total = Internal Timber Floor Build (Attic) - Timber boards+joist = - Ceiling and services = Total = Timber Flat Roof - Asphalt waterproofing = - Timber joists and insulation = - Ceiling and services = Total = Timber Pitched Roof - Tiles = - Batens = - Rafters = Total = Roof Space - Joist+Insulation = - Soffit = Total = Masonry Walls - Blockwork = - Lightweight Blockwork 7.3N = - Standard clay bricks = Live Loadings Internal Live Loading - A1 Domestic use = - Partitions with a self-weight of 1.0 kN/m wall length: qk = 0.5 kN/m2. = Total = - Snow and access (flat roof) = - Snow and access (pitched roof) = - Flat roof accessible for normal maintenance and Repair - Category H = - Attic, live loading =				Output <table><tr><td>0.35</td><td>kN/m2</td></tr><tr><td>0.15</td><td>kN/m2</td></tr><tr><td>0.50</td><td>kN/m2</td></tr><tr><td>0.15</td><td>kN/m2</td></tr><tr><td>0.15</td><td>kN/m2</td></tr><tr><td>0.30</td><td>kN/m2</td></tr><tr><td>0.35</td><td>kN/m2</td></tr><tr><td>0.30</td><td>kN/m2</td></tr><tr><td>0.15</td><td>kN/m2</td></tr><tr><td>0.8</td><td>kN/m2</td></tr><tr><td>0.60</td><td>kN/m2</td></tr><tr><td>0.10</td><td>kN/m2</td></tr><tr><td>0.15</td><td>kN/m2</td></tr><tr><td>0.85</td><td>kN/m2</td></tr><tr><td>0.15</td><td>kN/m2</td></tr><tr><td>0.15</td><td>kN/m2</td></tr><tr><td>0.30</td><td>kN/m2</td></tr><tr><td>14.00</td><td>kN/m3</td></tr><tr><td>10.00</td><td>kN/m3</td></tr><tr><td>20.00</td><td>kN/m3</td></tr><tr><td>1.50</td><td>kN/m2</td></tr><tr><td>0.50</td><td>kN/m2</td></tr><tr><td>2.00</td><td>kN/m2</td></tr><tr><td>0.75</td><td>kN/m2</td></tr><tr><td>0.75</td><td>kN/m2</td></tr><tr><td>0.75</td><td>kN/m2</td></tr><tr><td>0.60</td><td>kN/m2</td></tr></table>		0.35	kN/m2	0.15	kN/m2	0.50	kN/m2	0.15	kN/m2	0.15	kN/m2	0.30	kN/m2	0.35	kN/m2	0.30	kN/m2	0.15	kN/m2	0.8	kN/m2	0.60	kN/m2	0.10	kN/m2	0.15	kN/m2	0.85	kN/m2	0.15	kN/m2	0.15	kN/m2	0.30	kN/m2	14.00	kN/m3	10.00	kN/m3	20.00	kN/m3	1.50	kN/m2	0.50	kN/m2	2.00	kN/m2	0.75	kN/m2	0.75	kN/m2	0.75	kN/m2	0.60	kN/m2
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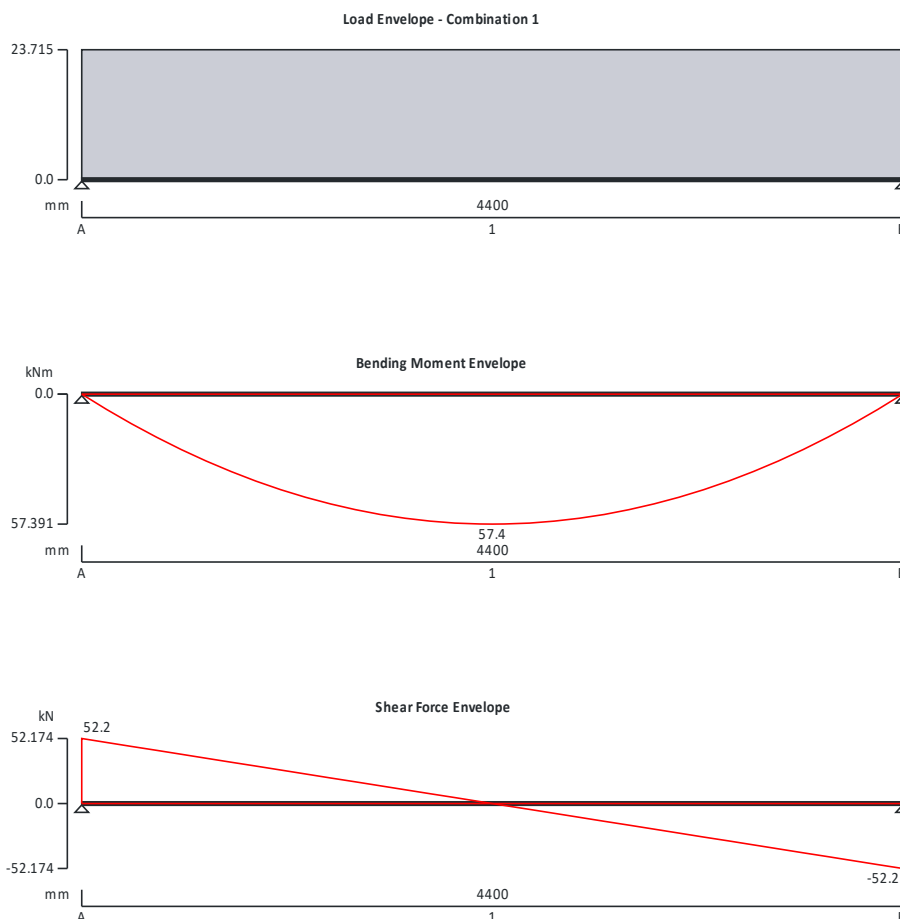
JUST BEAMS
28 HARROW ROAD, NEWPORT

Project MAES Y COED GUEST HOUSE, MERTHYR TYDFIL, CF47				Job Ref. 65	
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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 3.0.14



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 6 kN/m

Variable full UDL 10 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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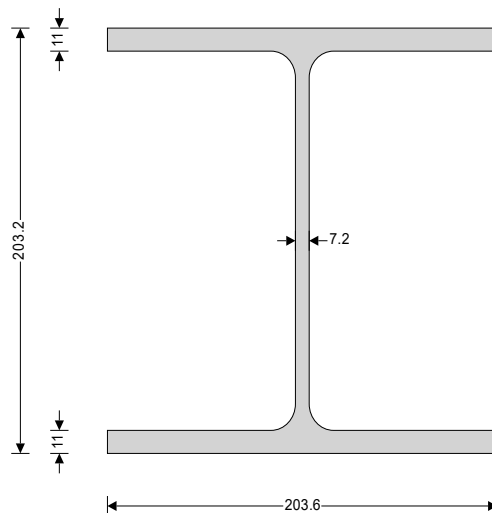
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Analysis results

Maximum moment	$M_{\max} = 57.4 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 52.2 \text{ kN}$	$V_{\min} = -52.2 \text{ kN}$
Deflection	$\delta_{\max} = 8.3 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 52.2 \text{ kN}$	$R_{A_{\min}} = 52.2 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 14.2 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 22 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 52.2 \text{ kN}$	$R_{B_{\min}} = 52.2 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 14.2 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 22 \text{ kN}$	

Section details

Section type	UC 203x203x46 (British Steel Section Range 2022 (BS4-1))
Steel grade	S355
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_f, t_w) = 11.0 \text{ mm}$
Nominal yield strength	$f_y = 355 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 470 \text{ N/mm}^2$
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

$$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$$

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

Width of section

$$c = d = 155.8 \text{ mm}$$

$$c / t_w = 26.6 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 85.5 \text{ mm}$$

$$c / t_f = 9.6 \times \varepsilon \leq 10 \times \varepsilon \quad \text{Class 2}$$

Section is class 2

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 181.2 \text{ mm}$$

Shear area factor

$$\eta = 1.000$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 52.2 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1802 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 369.3 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

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

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_{\max}}), \text{abs}(M_{s1_{\min}})) = 57.4 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 178.1 \text{ kNm}$$

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

$$g = \sqrt{[1 - (I_z / I_y)]} = 0.815$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained length

$$L = 1.0 \times L_{s1} = 4400 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 330.6 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.734$$

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

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.759$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.852$$



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Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.970}$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.878}$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{ply} \times f_y / \gamma_{M1} = \mathbf{156.5 \text{ kNm}}$

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 $\delta_{lim} = L_{s1} / 360 = \mathbf{12.2 \text{ mm}}$

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{8.305 \text{ mm}}$

PASS - Maximum deflection does not exceed deflection limit



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Project MAES Y COED GUEST HOUSE, MERTHYR TYDFIL, CF47				Job Ref. 65	
Section Wall				Sheet no./rev. 02/ 1	
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MASONRY WALL PANEL DESIGN

In accordance with BS5628-1:2005

Tedds calculation version 1.2.16

Masonry panel details

Unreinforced masonry wall without openings

Panel length $L = 5500$ mm

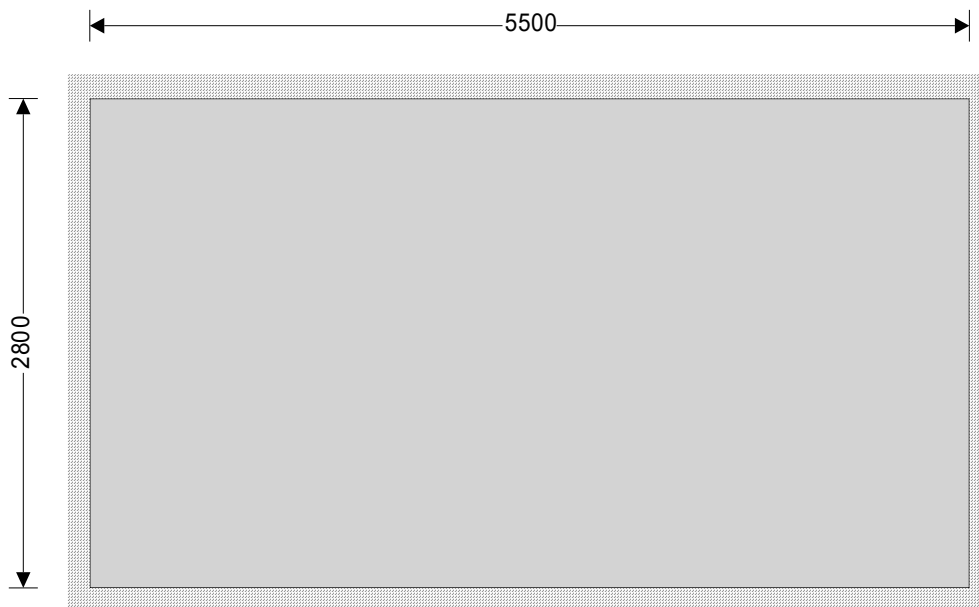
Panel height $h = 2800$ mm

Panel support conditions

All edges supported

Effective panel length $L_{ef} = 1.0 \times L = 5500$ mm

Effective panel height $h_{ef} = 1.0 \times h = 2800$ mm



Single-leaf wall construction details

Wall thickness $t = 215$ mm

Effective wall thickness $t_{ef} = t = 215$ mm



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Masonry details

Masonry type

Compressive strength of unit

Mortar strength Class/Designation

Height of masonry units

Density of masonry

Least horizontal dimension of masonry units

Ratio of height to least horizontal dimension

Aggregate concrete blocks with no voids

$p_{unit} = 7.3 \text{ N/mm}^2$

M6 / (ii)

$h_b = 215 \text{ mm}$

$\gamma = 18.0 \text{ kN/m}^3$

$t_{min} = 100 \text{ mm}$

$h_b / t_{min} = 2.15$

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

Characteristic compressive strength

$f_k = 6.40 \text{ N/mm}^2$

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

Plane of failure parallel to bed joints

$f_{kx_para} = 0.17 \text{ N/mm}^2$

Plane of failure perpendicular to bed joints

$f_{kx_perp} = 0.41 \text{ N/mm}^2$

Vertical loading details

Partial safety factors for material strength

Category of manufacturing control

Category II

Category of construction control

Normal

Partial safety factor for masonry in compression

$\gamma_{mc} = 3.50$

Partial safety factor for masonry in flexure

$\gamma_{mf} = 3.00$

Partial safety factor for masonry in shear

$\gamma_{mv} = 2.50$

Slenderness ratio (cl 24.1)

Allowable slenderness ratio

$SR_{all} = 27$

Slenderness ratio

$SR = \min(h_{ef}, L_{ef}) / t_{ef} = 13.0$

PASS - Slenderness ratio is less than maximum allowable

Vertical loading (cl 28)

Partial safety factors for design loads

Partial safety factor for design dead load

$\gamma_{FG} = 1.40$

Partial safety factor for design imposed load

$\gamma_{FQ} = 1.60$

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Check vertical loads at top of wall

Design vertical load on wall $F_v = (G_k + G_{ck} / w_b) \times \gamma_{fG} + (Q_k + Q_{ck} / w_b) \times \gamma_{fQ} = \mathbf{542.7 \text{ kN/m}}$

Design vertical load stress on wall $f_v = F_v / t = \mathbf{2.524 \text{ N/mm}^2}$

Bearing safety factor $\gamma_{bear} = \mathbf{1.50}$

Allowable stress capacity $f_{cap} = \gamma_{bear} \times f_k / \gamma_{mc} = \mathbf{2.743 \text{ N/mm}^2}$

PASS - Allowable stress capacity exceeds design vertical load stress on wall

Check vertical loads at $0.4 \times h$ below top of wall

Length of bearing distribution at $0.4 \times h$ $l_d = 0.8 \times h + w_b = \mathbf{2443 \text{ mm}}$

Eccentricity of concentrated load $e_c = (t - l_b) / 2 = \mathbf{57 \text{ mm}}$

Design vertical load on wall $F_{vm} = (G_k + G_{ck} / l_d) \times \gamma_{fG} + (Q_k + Q_{ck} / l_d) \times \gamma_{fQ} = \mathbf{45.1 \text{ kN/m}}$

Design vertical load stress on wall $f_{vm} = F_{vm} / t = \mathbf{0.210 \text{ N/mm}^2}$

Design bending moment $M_v = (G_k \times e_G + G_{ck} \times e_c / l_d) \times \gamma_{fG} + (Q_k \times e_Q + Q_{ck} \times e_c / l_d) \times \gamma_{fQ}$
 $M_v = \mathbf{2.6 \text{ kNm/m}}$

Resultant eccentricity at the top of the wall $e_x = M_v / F_{vm} = \mathbf{57 \text{ mm}}$

From BS5628-1 Table 7 - Capacity reduction factor

Capacity reduction factor $\beta = \mathbf{0.51}$

Allowable stress capacity $f_{capm} = \beta \times f_k / \gamma_{mc} = \mathbf{0.936 \text{ N/mm}^2}$

PASS - Allowable stress capacity exceeds design vertical load stress on wall



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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	104.3 kN	65.3 kN	1.11 N/mm ²	1.50 N/mm ²	0.742	Pass

Masonry panel details

Panel length	L = 2000 mm
Panel height	h = 2800 mm
Thickness of load bearing leaf	t = 215 mm
Effective height	$h_{ef} = \mathbf{2800}$ mm
Effective thickness	$t_{ef} = \mathbf{215}$ mm

Masonry material details

Unit type	Aggregate concrete - Group 1
Compressive strength of masonry unit	$f_c = \mathbf{3.6}$ N/mm ²
Height of unit	$h_u = \mathbf{215}$ mm
Width of unit	$w_u = \mathbf{100}$ mm
Conditioning factor	$k = \mathbf{1.0}$
- Conditioning to the air dry condition in accordance with cl.7.3.2	
Shape factor - Table A.1	$d_{sf} = \mathbf{1.38}$
Mean compressive strength of masonry unit	$f_b = f_c \times k \times d_{sf} = \mathbf{4.97}$ N/mm ²
Specific weight of units	$\gamma = \mathbf{18}$ kN/m ³
Mortar type	M4 - General Purpose
Compressive strength of mortar	$f_m = \mathbf{4.0}$ N/mm ²
Compressive strength factor - Tbl. NA 4	$K = \mathbf{0.75}$
Characteristic compressive strength - eq. 3.1	$f_k = K \times f_b^{0.7} \times f_m^{0.3} = \mathbf{3.49}$ N/mm ²
Short term secant modulus of elasticity factor	$K_E = \mathbf{1000}$
Modulus of elasticity - cl.3.7.2	$E_w = K_E \times f_k = \mathbf{3491}$ 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	$\gamma_M = \mathbf{3.00}$
Cross-sectional area of wall	$A = L \times t = \mathbf{0.43}$ m ²
Design compressive strength of masonry	$f_d = f_k / \gamma_M = \mathbf{1.16}$ N/mm ²

Partial safety factors for design loads

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

Superimposed vertical loading details

Permanent UDL at top of wall	$g_k = \mathbf{0.00}$ kN/m
Variable UDL at top of wall	$q_k = \mathbf{0.00}$ kN/m

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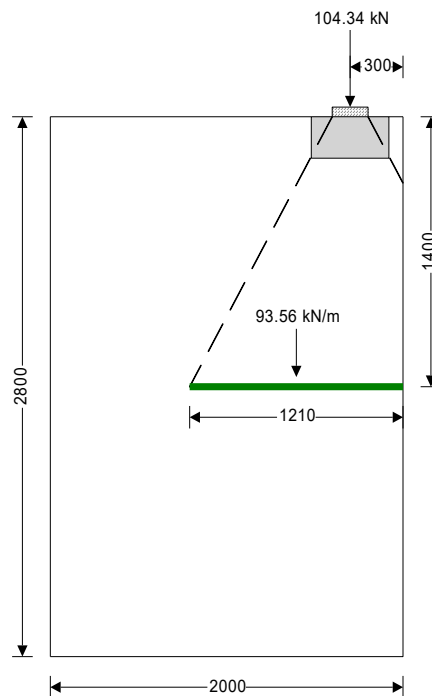
Eccentricity of permanent UDL load $e_{gu} = 0 \text{ mm}$
Eccentricity of variable UDL load $e_{qu} = 0 \text{ mm}$

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit $\lambda_{lim} = 27$
Slenderness ratio $\lambda = h_{ef} / t_{ef} = 13.0$

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load $G_{kc1} = 28.40 \text{ kN}$
Variable concentrated load $Q_{kc1} = 44.00 \text{ kN}$
Eccentricity of concentrated load $e_{c1} = 0 \text{ mm}$
Length of concentrated load $L_{c1} = 203 \text{ mm}$
Width of concentrated load $w_{c1} = 215 \text{ mm}$
Height of concentrated load $h_{c1} = 2800 \text{ mm}$
Distance of load to right vertical edge $r_{11} = 198 \text{ mm}$
Distance of load to nearest vertical edge $a_{11} = 198 \text{ mm}$

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 $A_{b1} = L_{c1} \times w_{c1} = 43645 \text{ mm}^2$
Effective length of bearing at mid-height $l_{efm1} = L_{c1} + h_{c1} / 2 \times \tan(30) + r_{11} = 1210 \text{ mm}$
Effective bearing area $A_{ef1} = l_{efm1} \times t = 260105 \text{ mm}^2$
Bearing area ratio check $A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = 0.17$
Initial enhancement factor $\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = 1.34$
Maximum enhancement factor $\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = 1.29$
Enhancement factor for concentrated loads $\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1}) = 1.29$



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Design value of the concentrated load

$$N_{Edc1} = G_{kc1} \times \gamma_{FG} + Q_{kc1} \times \gamma_{FQ} = \mathbf{104.34 \text{ kN}}$$

Design value concentrated load resistance

$$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d = \mathbf{65.29 \text{ kN}}$$

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

$$P_{11} = \mathbf{220 \text{ mm}}$$

Length of spreader

$$L_{sp1} = \mathbf{440 \text{ mm}}$$

Height of spreader

$$h_{sp1} = \mathbf{215 \text{ mm}}$$

Width of spreader

$$w_{sp1} = \mathbf{215 \text{ mm}}$$

Eccentricity of load on spreader

$$e_{sp1} = \mathbf{0 \text{ mm}}$$

Modulus of elasticity

$$E_{sp1} = \mathbf{29962 \text{ N/mm}^2}$$

Second moment of area

$$I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3 = \mathbf{178062552 \text{ mm}^4}$$

Modulus of the wall

$$k_0 = E_w / h = \mathbf{1.25 \text{ N/mm}^2/\text{mm}}$$

Winkler's constant

$$K_{c1} = k_0 \times w_{sp1} = \mathbf{268.09 \text{ N/mm/mm}}$$

Characteristic of the system

$$\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4} = \mathbf{0.00188 \text{ mm}^{-1}}$$

Classification of spreader

$$\alpha L_1 = \alpha_1 \times L_{sp1} = \mathbf{0.83 \text{ Medium}}$$

Krilov's functions for the spreader length

$$B_{\alpha 1} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = \mathbf{0.82}$$

$$C_{\alpha 1} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) = \mathbf{0.34}$$

$$D_{\alpha 1} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = \mathbf{0.09}$$

Krilov's functions at the point load

$$A_{\alpha P11} = \cosh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi) = \mathbf{1.00}$$

$$B_{\alpha P11} = 1/2 \times (\cosh(\alpha_1 \times P_{11}) \times \sin(180 \times \alpha_1 \times P_{11} / \pi) + \sinh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi)) = \mathbf{0.41}$$

Using method of initial conditions

Initial moment of LH edge

$$M_{01} = \mathbf{0 \text{ kNm}}$$

Initial shear of LH edge

$$V_{01} = \mathbf{0 \text{ kN}}$$

Which gives

$$(4 \times \alpha_1^2 \times C_{\alpha 1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha 1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - B_{\alpha P11} / \alpha_1 \times N_{Edc1} = \mathbf{0.00 \text{ kNm}}$$

and

$$(4 \times \alpha_1^3 \times B_{\alpha 1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha 1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - A_{\alpha P11} \times N_{Edc1} = \mathbf{0.00 \text{ kN}}$$

Therefore,

Initial deflection of LH edge

$$\delta_{01} = \mathbf{0.87675 \text{ mm}}$$

Initial rotation of LH edge

$$\Phi_{01} = \mathbf{0.000079}$$

Location of maximum deflection

$$x_{def1} = \mathbf{220 \text{ mm}}$$

Krilov's functions at the spreader length

$$A_{\alpha x_{def1}} = \cosh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi) = \mathbf{1.00}$$

$$B_{\alpha x_{def1}} = 1/2 \times (\cosh(\alpha_1 \times x_{def1}) \times \sin(180 \times \alpha_1 \times x_{def1} / \pi) + \sinh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi)) = \mathbf{0.41}$$

Distance of point load right of location

$$p_{1def1} = \mathbf{0 \text{ mm}}$$

Krilov's functions at the spreader length

$$D_{\alpha p_{1def1}} = 1/4 \times (\cosh(\alpha_1 \times p_{1def1}) \times \sin(180 \times \alpha_1 \times p_{1def1} / \pi) - \sinh(\alpha_1 \times p_{1def1}) \times \cos(180 \times \alpha_1 \times p_{1def1} / \pi)) = \mathbf{0.00}$$



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Particular integral due to load

$$\delta'_1 = D_{op1def1} / \alpha_1^3 \times N_{Edc1} / (I_{sp1} \times E_{sp1}) = \mathbf{0.000 \text{ mm}}$$

Maximum deflection

$$\delta_{max1} = A_{oxdef1} \times \delta_{01} + B_{oxdef1} \times \Phi_{01} / \alpha_1 + \delta'_1 = \mathbf{0.890 \text{ mm}}$$

Location of maximum moment

$$x_{M1} = \mathbf{220 \text{ mm}}$$

Krilov's functions at the spreader length

$$C_{oxM1} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) = \mathbf{0.09}$$

$$D_{oxM1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi)) = \mathbf{0.01}$$

Distance of point load right of loaction

$$p_{1M1} = \mathbf{0 \text{ mm}}$$

Krilov's functions at the spreader length

$$B_{op1M1} = 1/2 \times (\cosh(\alpha_1 \times p_{1M1}) \times \sin(180 \times \alpha_1 \times p_{1M1} / \pi) + \sinh(\alpha_1 \times p_{1M1}) \times \cos(180 \times \alpha_1 \times p_{1M1} / \pi)) = \mathbf{0.00}$$

Particular integral due to load

$$M'_1 = -B_{op1M1} / \alpha_1 \times N_{Edc1} = \mathbf{0.00 \text{ kNm}}$$

Maximum moment

$$M_{Edsp1} = (4 \times \alpha_1^2 \times C_{oxM1} \times \delta_{01} + 4 \times \alpha_1 \times D_{oxM1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M'_1 = \mathbf{5.72 \text{ kNm}}$$

Location of maximum shear

$$x_{V1} = \mathbf{220 \text{ mm}}$$

Krilov's functions at the spreader length

$$B_{oxV1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi)) = \mathbf{0.41}$$

$$C_{oxV1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) = \mathbf{0.09}$$

Distance of point load right of loaction

$$p_{1V1} = \mathbf{0 \text{ mm}}$$

Krilov's functions at the spreader length

$$A_{op1V1} = \cosh(\alpha_1 \times p_{1V1}) \times \cos(180 \times \alpha_1 \times p_{1V1} / \pi) = \mathbf{1.00}$$

Particular integral due to load

$$V'_1 = -A_{op1V1} \times N_{Edc1} = \mathbf{-104.34 \text{ kN}}$$

Shear at concentrated point load

$$V_1 = (4 \times \alpha_1^3 \times B_{oxV1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{oxV1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + V'_1 = \mathbf{-52.17 \text{ kN}}$$

Maximum shear

$$V_{Edsp1} = \text{Max}(\text{Abs}(V_1), N_{Edc1} - \text{Abs}(V_1)) = \mathbf{52.17 \text{ kN}}$$

Maximum allowable stress under spreader

$$\sigma_{Rdsp1} = \beta_1 \times f_d = \mathbf{1.50 \text{ N/mm}^2}$$

Maximum reaction

$$N_{Edsp1} = K_{c1} \times \delta_{max1} = \mathbf{238.53 \text{ kN/m}}$$

Design stress

$$\sigma_{Edsp1} = N_{Edsp1} / W_{sp1} = \mathbf{1.11 \text{ N/mm}^2}$$

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

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)

$$e_{init} = h_{ef} / 450 = \mathbf{6.2 \text{ mm}}$$

Concentrated load at mid-height as UDL

$$N_{mc1} = N_{Edc1} / l_{efm1} = \mathbf{86.25 \text{ kN/m}}$$

Vertical load at mid-height

$$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = \mathbf{93.56 \text{ kN/m}}$$

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

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = \mathbf{6.2 \text{ mm}}$$

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

$$e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \times t) = \mathbf{10.8 \text{ mm}}$$



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From eq. G2

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

From eq. G3

$$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = \mathbf{0.52}$$

Capacity reduction factor - eq. G1

$$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) = \mathbf{0.79}$$

Design vertical resistance of panel - eq.6.2

$$N_{Rd1} = \Phi_{m1} \times t \times f_d = \mathbf{196.77 \text{ kN/m}}$$

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	52.2 kN	31.2 kN	1.19 N/mm ²	1.54 N/mm ²	0.776	Pass

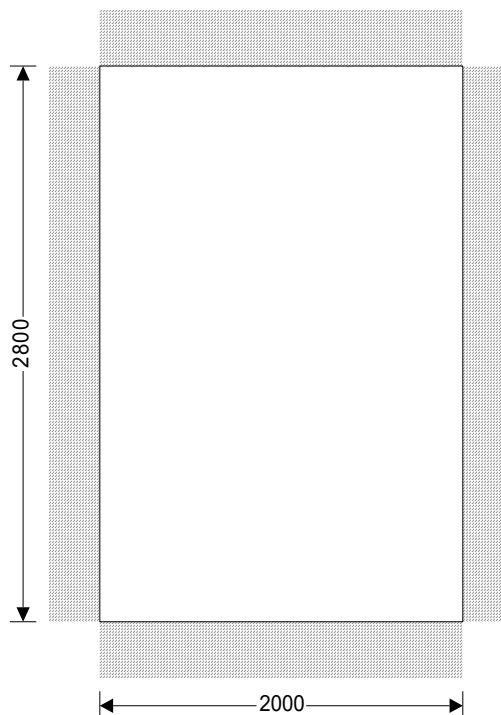
Masonry panel details

Panel length L = 2000 mm

Panel height h = 2800 mm

Panel support conditions

All edges supported



Effective height of masonry wall - Section 5.5.1.2

Vertical restraints

$L < 30 \times t_{ef}$ (Wall restrained at the top and bottom and stiffened on two vertical edges)

Horizontal restraints

Simply supported

Reduction factor - eq. 5.9

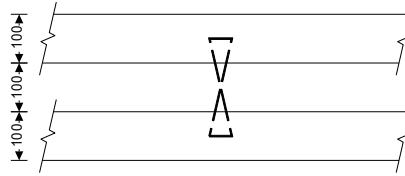
$\rho_4 = (0.5 \times L) / h = 0.36$

Effective height of wall - eq. 5.2

$h_{ef} = \rho_4 \times h = 1000 \text{ mm}$

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Wall construction details



Wall type	Cavity wall
Inner leaf thickness	$t_2 = 100.0$ mm
Cavity thickness	$t_c = 100.0$ mm
Outer leaf thickness	$t_1 = 100.0$ mm

Effective thickness of masonry wall - Section 5.5.1.3

Effective thickness of inner leaf	$t_{ef2} = t_2 = 100.0$ mm
Effective thickness of outer leaf	$t_{ef1} = \text{Min}(t_1, t_2) = 100.0$ mm
Factor for the relative E values	$k_{lef} = 1.00$
Effective thickness - eq. 5.11	$t_{ef} = (k_{lef} \times t_{ef1}^3 + t_{ef2}^3)^{1/3} = 126.0$ 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	$\gamma = 18$ 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	$f_k = K \times f_b^{0.7} \times f_m^{0.3} = 3.49$ 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 for inner (loaded) leaf

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	$A = L \times t = 0.20$ 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	$\gamma_{FG} = 1.35$
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
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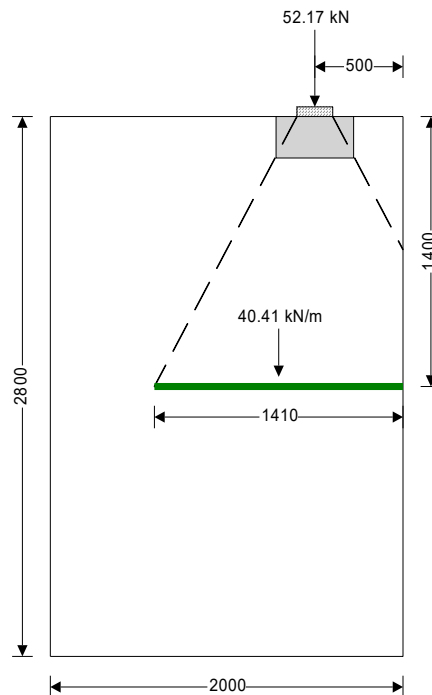
Variable UDL at top of wall	$q_k = 0.00 \text{ kN/m}$
Eccentricity of permanent UDL load	$e_{gu} = 0 \text{ mm}$
Eccentricity of variable UDL load	$e_{qu} = 0 \text{ mm}$

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit	$\lambda_{\text{lim}} = 27$
Slenderness ratio	$\lambda = h_{\text{ef}} / t_{\text{ef}} = 7.9$

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load	$G_{kc1} = 14.20$ kN
Variable concentrated load	$Q_{kc1} = 22.00$ kN
Eccentricity of concentrated load	$e_{c1} = 0$ mm
Length of concentrated load	$L_{c1} = 203$ mm
Width of concentrated load	$w_{c1} = 100$ mm
Height of concentrated load	$h_{c1} = 2800$ mm
Distance of load to right vertical edge	$r_{11} = 398$ mm
Distance of load to nearest vertical edge	$a_{11} = 398$ 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	$A_{b1} = L_{c1} \times w_{c1} = \mathbf{20300 \text{ mm}^2}$
Effective length of bearing at mid-height	$l_{efm1} = L_{c1} + h_{c1} / 2 \times \tan(30) + r_{11} = \mathbf{1410 \text{ mm}}$
Effective bearing area	$A_{ef1} = l_{efm1} \times t = \mathbf{140979 \text{ mm}^2}$
Bearing area ratio check	$A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = \mathbf{0.14}$
Initial enhancement factor	$\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = \mathbf{1.40}$
Maximum enhancement factor	$\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = \mathbf{1.32}$



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Enhancement factor for concentrated loads

Design value of the concentrated load

Design value concentrated load resistance

$$\beta_1 = \text{Min}(\beta_{\text{init}1}, \beta_{\text{max}1}) = \mathbf{1.32}$$

$$N_{\text{Edc}1} = G_{\text{kc}1} \times \gamma_{\text{fG}} + Q_{\text{kc}1} \times \gamma_{\text{fQ}} = \mathbf{52.17 \text{ kN}}$$

$$N_{\text{Rdc}1} = \beta_1 \times A_{\text{b}1} \times f_{\text{d}} = \mathbf{31.21 \text{ kN}}$$

Applied concentrated load exceeds design resistance, spreader required!

Design of spreader beam

Type of spreader

Type of bearing onto spreader

Location of load from RHS of spreader

Length of spreader

Height of spreader

Width of spreader

Eccentricity of load on spreader

Modulus of elasticity

Second moment of area

Modulus of the wall

Winkler's constant

Characteristic of the system

Classification of spreader

Krilov's functions for the spreader length

Concrete padstone

Point load

$$P_{11} = \mathbf{220 \text{ mm}}$$

$$L_{\text{sp}1} = \mathbf{440 \text{ mm}}$$

$$h_{\text{sp}1} = \mathbf{215 \text{ mm}}$$

$$w_{\text{sp}1} = \mathbf{100 \text{ mm}}$$

$$e_{\text{sp}1} = \mathbf{0 \text{ mm}}$$

$$E_{\text{sp}1} = \mathbf{29962 \text{ N/mm}^2}$$

$$I_{\text{sp}1} = 1/12 \times w_{\text{sp}1} \times h_{\text{sp}1}^3 = \mathbf{82819792 \text{ mm}^4}$$

$$k_0 = E_w / h = \mathbf{1.25 \text{ N/mm}^2/\text{mm}}$$

$$K_{\text{c}1} = k_0 \times w_{\text{sp}1} = \mathbf{124.69 \text{ N/mm/mm}}$$

$$\alpha_1 = (K_{\text{c}1} / (4 \times E_{\text{sp}1} \times I_{\text{sp}1}))^{1/4} = \mathbf{0.00188 \text{ mm}^{-1}}$$

$$\alpha L_1 = \alpha_1 \times L_{\text{sp}1} = \mathbf{0.83 \text{ Medium}}$$

$$B_{\alpha 1} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = \mathbf{0.82}$$

$$C_{\alpha 1} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) = \mathbf{0.34}$$

$$D_{\alpha 1} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = \mathbf{0.09}$$

Krilov's functions at the point load

$$A_{\alpha P11} = \cosh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi) = \mathbf{1.00}$$

$$B_{\alpha P11} = 1/2 \times (\cosh(\alpha_1 \times P_{11}) \times \sin(180 \times \alpha_1 \times P_{11} / \pi) + \sinh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi)) = \mathbf{0.41}$$

Using method of initial conditions

Initial moment of LH edge

Initial shear of LH edge

Which gives

$$M_{01} = \mathbf{0 \text{ kNm}}$$

$$V_{01} = \mathbf{0 \text{ kN}}$$

$$(4 \times \alpha_1^2 \times C_{\alpha 1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha 1} \times \Phi_{01}) \times E_{\text{sp}1} \times I_{\text{sp}1} - B_{\alpha P11} / \alpha_1 \times N_{\text{Edc}1} = \mathbf{0.00 \text{ kNm}}$$

and

$$(4 \times \alpha_1^3 \times B_{\alpha 1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha 1} \times \Phi_{01}) \times E_{\text{sp}1} \times I_{\text{sp}1} - A_{\alpha P11} \times N_{\text{Edc}1} = \mathbf{0.00 \text{ kN}}$$

Therefore,

Initial deflection of LH edge

$$\delta_{01} = \mathbf{0.94250 \text{ mm}}$$

Initial rotation of LH edge

$$\Phi_{01} = \mathbf{0.000084}$$

Location of maximum deflection

$$x_{\text{def}1} = \mathbf{220 \text{ mm}}$$

Krilov's functions at the spreader length

$$A_{\alpha x\text{def}1} = \cosh(\alpha_1 \times x_{\text{def}1}) \times \cos(180 \times \alpha_1 \times x_{\text{def}1} / \pi) = \mathbf{1.00}$$

$$B_{\alpha x\text{def}1} = 1/2 \times (\cosh(\alpha_1 \times x_{\text{def}1}) \times \sin(180 \times \alpha_1 \times x_{\text{def}1} / \pi) + \sinh(\alpha_1 \times x_{\text{def}1}) \times \cos(180 \times \alpha_1 \times x_{\text{def}1} / \pi)) = \mathbf{0.41}$$

Distance of point load right of loaction

$$p_{1\text{def}1} = \mathbf{0 \text{ mm}}$$

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Krilov's functions at the spreader length

$$D_{op1def1} = 1/4 \times (\cosh(\alpha_1 \times p_{1def1}) \times \sin(180 \times \alpha_1 \times p_{1def1} / \pi) - \sinh(\alpha_1 \times p_{1def1}) \times \cos(180 \times \alpha_1 \times p_{1def1} / \pi)) = \mathbf{0.00}$$

Particular integral due to load

$$\delta'_1 = D_{op1def1} / \alpha_1^3 \times N_{Edc1} / (I_{sp1} \times E_{sp1}) = \mathbf{0.000 \text{ mm}}$$

Maximum deflection

$$\delta_{max1} = A_{oxdef1} \times \delta_{01} + B_{oxdef1} \times \Phi_{01} / \alpha_1 + \delta'_1 = \mathbf{0.956 \text{ mm}}$$

Location of maximum moment

$$x_{M1} = \mathbf{220 \text{ mm}}$$

Krilov's functions at the spreader length

$$C_{oxM1} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) = \mathbf{0.09}$$

$$D_{oxM1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi)) = \mathbf{0.01}$$

Distance of point load right of loaction

$$p_{1M1} = \mathbf{0 \text{ mm}}$$

Krilov's functions at the spreader length

$$B_{op1M1} = 1/2 \times (\cosh(\alpha_1 \times p_{1M1}) \times \sin(180 \times \alpha_1 \times p_{1M1} / \pi) + \sinh(\alpha_1 \times p_{1M1}) \times \cos(180 \times \alpha_1 \times p_{1M1} / \pi)) = \mathbf{0.00}$$

Particular integral due to load

$$M'_1 = -B_{op1M1} / \alpha_1 \times N_{Edc1} = \mathbf{0.00 \text{ kNm}}$$

Maximum moment

$$M_{Edsp1} = (4 \times \alpha_1^2 \times C_{oxM1} \times \delta_{01} + 4 \times \alpha_1 \times D_{oxM1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M'_1 = \mathbf{2.86 \text{ kNm}}$$

Location of maximum shear

$$x_{V1} = \mathbf{220 \text{ mm}}$$

Krilov's functions at the spreader length

$$B_{oxV1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi)) = \mathbf{0.41}$$

$$C_{oxV1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) = \mathbf{0.09}$$

Distance of point load right of loaction

$$p_{1V1} = \mathbf{0 \text{ mm}}$$

Krilov's functions at the spreader length

$$A_{op1V1} = \cosh(\alpha_1 \times p_{1V1}) \times \cos(180 \times \alpha_1 \times p_{1V1} / \pi) = \mathbf{1.00}$$

Particular integral due to load

$$V'_1 = -A_{op1V1} \times N_{Edc1} = \mathbf{-52.17 \text{ kN}}$$

Shear at concentrated point load

$$V_1 = (4 \times \alpha_1^3 \times B_{oxV1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{oxV1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + V'_1 = \mathbf{-26.09 \text{ kN}}$$

Maximum shear

$$V_{Edsp1} = \text{Max}(\text{Abs}(V_1), N_{Edc1} - \text{Abs}(V_1)) = \mathbf{26.09 \text{ kN}}$$

Maximum allowable stress under spreader

$$\sigma_{Rdsp1} = \beta_1 \times f_d = \mathbf{1.54 \text{ N/mm}^2}$$

Maximum reaction

$$N_{Edsp1} = K_{c1} \times \delta_{max1} = \mathbf{119.26 \text{ kN/m}}$$

Design stress

$$\sigma_{Edsp1} = N_{Edsp1} / W_{sp1} = \mathbf{1.19 \text{ N/mm}^2}$$

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

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)

$$e_{init} = h_{ef} / 450 = \mathbf{2.2 \text{ mm}}$$

Concentrated load at mid-height as UDL

$$N_{mc1} = N_{Edc1} / l_{efm1} = \mathbf{37.01 \text{ kN/m}}$$

Vertical load at mid-height

$$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = \mathbf{40.41 \text{ kN/m}}$$

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

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = \mathbf{2.2 \text{ mm}}$$

Slenderness ratio limit for creep eccentricity

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



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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

$$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = \mathbf{0.28}$$

Capacity reduction factor - eq. G1

$$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) = \mathbf{0.87}$$

Design vertical resistance of panel - eq.6.2

$$N_{Rd1} = \Phi_{m1} \times t \times f_d = \mathbf{100.72 \text{ kN/m}}$$

PASS - Design value of vertical resistance exceeds applied vertical load

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RC RAFT FOUNDATION

In accordance with EN1992-1-1:2004 incorporating Corrigenda January 2008 and the UK national annex

Tedds calculation version 1.0.05

Design summary

Overall design status **PASS**

Overall design utilisation **0.942**

Slab

Description	Unit	Provided	Required	Utilisation	Result
Bearing load	kN/m ²	50.0	11.7	0.234	PASS
Top reinforcement	mm ² /m	393	165	0.420	PASS
Bottom reinforcement	mm ² /m	393	165	0.420	PASS
Shear resistance	kN/m	56.0	10.5	0.188	PASS
Span to depth ratio		60.0	32.9	0.548	PASS

Edge/corner beam

Description	Unit	Provided	Required	Utilisation	Result
Top slab reinf. for bearing	mm ² /m	393	283	0.720	PASS
Top reinforcement (edge)	mm ²	603	568	0.942	PASS
Bottom reinforcement (edge)	mm ²	603	568	0.942	PASS
Shear reinforcement (edge)	mm ² /m	698	362	0.519	PASS
Boot reinforcement (edge boot)	mm ² /m	349	291	0.833	PASS
Shear resistance (edge boot)	kN/m	103.6	21.2	0.205	PASS
Bearing pressure (corner boot)	kN/m ²	50	15	0.296	PASS
Top reinforcement (corner)	mm ²	603	387	0.641	PASS
Shear resistance (corner)	kN	74.9	55.8	0.745	PASS
Span to depth ratio (corner)		16.0	6.6	0.415	PASS

Internal beam

Description	Unit	Provided	Required	Utilisation	Result
Top slab reinf. for transfer	mm ² /m	393	157	0.400	PASS
Bot slab reinf. for transfer	mm ² /m	393	157	0.400	PASS
Bot slab reinf. for bearing	mm ² /m	393	157	0.400	PASS
Top reinforcement	mm ²	804	635	0.789	PASS
Bottom reinforcement	mm ²	804	635	0.789	PASS
Shear reinforcement	mm ² /m	698	407	0.583	PASS

Concrete details - Table 3.1. Strength and deformation characteristics for concrete

Concrete strength class	C32/40
Aggregate type	Quartzite
Aggregate adjustment factor - cl.3.1.3(2)	AAF = 1.0
Characteristic compressive cylinder strength	$f_{ck} = \mathbf{32 \text{ N/mm}^2}$
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = \mathbf{40 \text{ N/mm}^2}$
Mean value of axial tensile strength	$f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck} / 1 \text{ N/mm}^2)^{2/3} = \mathbf{3.0 \text{ N/mm}^2}$
Secant modulus of elasticity of concrete	$E_{cm} = 22 \text{ kN/mm}^2 \times (f_{cm} / 10 \text{ N/mm}^2)^{0.3} \times \text{AAF} = \mathbf{33346 \text{ N/mm}^2}$
Ultimate strain - Table 3.1	$\epsilon_{cu2} = \mathbf{0.0035}$
Shortening strain - Table 3.1	$\epsilon_{cu3} = \mathbf{0.0035}$
Effective compression zone height factor	$\lambda = \mathbf{0.80}$
Effective strength factor	$\eta = \mathbf{1.00}$



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Coefficient k_1	$k_1 = 0.40$
Coefficient k_2	$k_2 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2}) = 1.00$
Coefficient k_3	$k_3 = 0.40$
Coefficient k_4	$k_4 = 1.0 \times (0.6 + 0.0014 / \varepsilon_{cu2}) = 1.00$
Partial factor for concrete - Table 2.1N	$\gamma_C = 1.50$
Compressive strength coefficient - cl.3.1.6(1)	$\alpha_{cc} = 0.85$
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = 18.1 \text{ N/mm}^2$
Compressive strength coefficient - cl.3.1.6(1)	$\alpha_{csw} = 1.00$
Design compressive concrete strength - exp.3.15	$f_{cwd} = \alpha_{csw} \times f_{ck} / \gamma_C = 21.3 \text{ N/mm}^2$
Maximum aggregate size	$h_{agg} = 20 \text{ mm}$
Monolithic simple support moment factor	$\beta_1 = 0.25$
Density of reinforced concrete	$\rho_{conc} = 24.5 \text{ kN/m}^3$

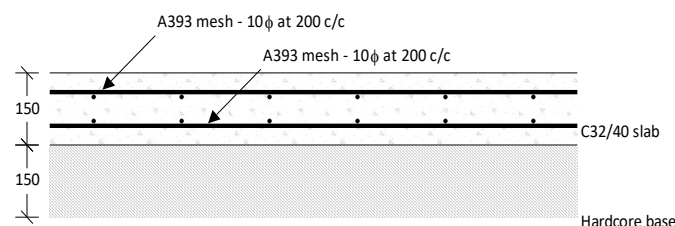
Reinforcement details

Characteristic yield strength of reinforcement	$f_{yk} = 500 \text{ N/mm}^2$
Partial factor for reinforcing steel - Table 2.1N	$\gamma_S = 1.15$
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S = 435 \text{ N/mm}^2$

Soil properties

Allowable bearing pressure	$q_{allow} = 50.0 \text{ kN/m}^2$
Soil classification	D - Two or more variable soil types with variable density
Density of hardcore/compacted fill	$\rho_{fill} = 20 \text{ kN/m}^3$
Angle of dispersal through fill from horizontal	$\alpha_{fill} = 60.0 \text{ deg}$
Assumed diameter of depression	$\phi_{dep_basic} = 3500 \text{ mm}$

Slab details



Section through slab

Slab thickness	$h_{slab} = 150 \text{ mm}$
Hardcore thickness	$h_{fill_slab} = 150 \text{ mm}$
Diameter of depression modified for hardcore	$\phi_{dep_slab} = \phi_{dep_basic} - h_{fill_slab_b1} = 3350 \text{ mm}$
Nominal cover to slab top reinforcement	$C_{nom_slab_t} = 35 \text{ mm}$
Nominal cover to slab bottom reinforcement	$C_{nom_slab_b} = 35 \text{ mm}$
Top reinforcement provided	A393 mesh - 10φ at 200 c/c
Area of top reinforcement provided	$A_{s,prov_t_slab} = 393 \text{ mm}^2/\text{m}$
Average effective depth of top reinforcement	$d_{avg_t_slab} = h_{slab} - C_{nom_slab_t} - \phi_{t_slab} = 105 \text{ mm}$
Minimum effective depth of top reinforcement	$d_{min_t_slab} = h_{slab} - C_{nom_slab_t} - 1.5 \times \phi_{t_slab} = 100 \text{ mm}$
Bottom reinforcement provided	A393 mesh - 10φ at 200 c/c
Area of bottom reinforcement provided	$A_{s,prov_b_slab} = 393 \text{ mm}^2/\text{m}$
Average effective depth of bottom reinforcement	$d_{avg_b_slab} = h_{slab} - C_{nom_slab_b} - \phi_{b_slab} = 105 \text{ mm}$

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Minimum effective depth of bottom reinforcement $d_{min_b_slab} = h_{slab} - C_{nom_slab_b} - 1.5 \times \phi_{b_slab} = 100 \text{ mm}$

Slab UDL loading

Permanent UDL to slab

$$W_{G_slab} = 2.0 \text{ kN/m}^2$$

Variable UDL to slab

$$W_{Q_slab} = 3.0 \text{ kN/m}^2$$

Slab design check

Slab self weight

$$W_{slab} = \rho_{conc} \times h_{slab} = 3.7 \text{ kN/m}^2$$

Slab fill self weight

$$W_{fill_slab} = \rho_{fill} \times h_{fill_slab} = 3.0 \text{ kN/m}^2$$

Total uniform load at formation level

$$F_{slab} = W_{slab} + W_{fill_slab} + W_{G_slab} + W_{Q_slab} = 11.7 \text{ kN/m}^2$$

PASS - allowable bearing pressure exceeds bearing pressure at formation level

Self weight and UDL forces in slab

Effective span of slab

$$l_{eff_slab} = \phi_{dep_slab} + d_{avg_t_slab} = 3455 \text{ mm}$$

Ultimate applied self weight and UDL's

$$F_{ult_slab} = 1.35 \times (W_{slab} + W_{G_slab}) + 1.50 \times W_{Q_slab} = 12.2 \text{ kN/m}^2$$

Moment at edge of depression

$$M_{neg_slab} = F_{ult_slab} \times l_{eff_slab}^2 / 32 = 4.5 \text{ kNm/m}$$

Moment at centre of depression

$$M_{pos_slab} = F_{ult_slab} \times l_{eff_slab}^2 \times (1 + \nu) / 64 = 2.7 \text{ kNm/m}$$

Shear force at edge of depression

$$V_{slab} = F_{ult_slab} \times l_{eff_slab} / 4 = 10.5 \text{ kN/m}$$

Total forces in slab

Moment at edge of depression

$$M_{neg_total_slab} = M_{neg_slab} = 4.5 \text{ kNm/m}$$

Moment at centre of depression

$$M_{pos_total_slab} = M_{pos_slab} = 2.7 \text{ kNm/m}$$

Shear force at edge of depression

$$V_{total_slab} = V_{slab} = 10.5 \text{ kN/m}$$

Reinforcement required in top of slab for bending

$$K_{total_t_slab} = M_{neg_total_slab} / (d_{avg_t_slab}^2 \times f_{ck}) = 0.013$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = 0.207$$

Lever arm

$$Z_{total_t_slab} = \min(0.5 \times d_{avg_t_slab} \times [1 + (1 - 2 \times K_{total_t_slab} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{avg_t_slab}) = 100 \text{ mm}$$

Minimum area of reinforcement required

$$A_{s,min_total_t_slab} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{avg_t_slab} = 165 \text{ mm}^2/\text{m}$$

Area of tension reinforcement required

$$A_{s,req_total_t_slab} = M_{neg_total_slab} / (f_{yd} \times Z_{total_t_slab}) = 105 \text{ mm}^2/\text{m}$$

PASS - Area of reinforcement provided exceeds area of reinforcement required

Reinforcement required in bottom of slab for bending

$$K_{total_b_slab} = M_{pos_total_slab} / (d_{avg_b_slab}^2 \times f_{ck}) = 0.008$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = 0.207$$

Lever arm

$$Z_{total_b_slab} = \min(0.5 \times d_{avg_b_slab} \times [1 + (1 - 2 \times K_{total_b_slab} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{avg_b_slab}) = 100 \text{ mm}$$

Minimum area of reinforcement required

$$A_{s,min_total_b_slab} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{avg_b_slab} = 165 \text{ mm}^2/\text{m}$$

Area of tension reinforcement required

$$A_{s,req_total_b_slab} = M_{pos_total_slab} / (f_{yd} \times Z_{total_b_slab}) = 63 \text{ mm}^2/\text{m}$$

PASS - Area of reinforcement provided in top to span local depression is adequate

Shear resistance with no shear reinforcement

Strength reduction factor

$$v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2) = 0.523$$

Max allowable design shear force (cl.6.2.2)

$$V_{Ed,max_slab} = 0.5 \times d_{min_t_slab} \times v_1 \times f_{cwd} = 558 \text{ kN/m}$$

PASS - Design shear force in slab is less than the maximum allowable shear force

Reinforcement ratio

$$\rho_l_slab = \min(A_{s,prov_t_slab} / d_{min_t_slab}, 0.02) = 0.00393$$



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k coefficient

C_{Rd} coefficient

Minimum shear stress

Minimum design shear resistance

Design shear resistance

$$k_{v_slab} = \min(1 + (200\text{mm} / d_{min_t_slab})^{0.5}, 2) = \mathbf{2.000}$$

$$C_{Rd,c_slab} = 0.18 / \gamma_c = \mathbf{0.120}$$

$$V_{min_slab} = 0.035 \text{ N/mm}^2 \times k_{v_slab}^{3/2} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} = \mathbf{0.560 \text{ N/mm}^2}$$

$$V_{Rd,c,min_slab} = V_{min_slab} \times d_{min_t_slab} = \mathbf{56.0 \text{ kN/m}}$$

$$V_{Rd,c_slab} = \max(C_{Rd,c_slab} \times k_{v_slab} \times 1 \text{ N/mm}^2 \times (100 \times \rho_{l_slab} \times f_{ck} / 1 \text{ N/mm}^2)^{1/3} \times d_{min_t_slab}, V_{Rd,c,min_slab}) = \mathbf{56.0 \text{ kN/m}}$$

PASS - Design shear resistance exceeds design shear

Deflection control - Section 7.4

Reference reinforcement ratio

Required tension reinforcement ratio

Required compression reinforcement ratio

Structural system factor - Table 7.4N

Basic allowable span to depth ratio

Reinforcement factor - exp.7.17

Flange width factor

Long span supporting brittle partition factor

Allowable span to depth ratio

Actual span to depth ratio

$$\rho_{0_slab} = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 = \mathbf{0.00566}$$

$$\rho_{slab} = A_{s,req_total_b_slab} / d_{min_b_slab} = \mathbf{0.00063}$$

$$\rho'_{slab} = \mathbf{0}$$

$$K_{b,defl_slab} = \mathbf{1.5}$$

$$\delta_{basic_slab} = K_{b,defl_slab} \times [11 + 1.5 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times \rho_{0_slab} / \rho_{slab} + 3.2 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times (\rho_{0_slab} / \rho_{slab} - 1)^{1.5}] = \mathbf{747.155}$$

$$K_{s_slab} = \min(A_{s,prov_b_slab} / A_{s,req_total_b_slab} \times 500 \text{ N/mm}^2 / f_{yk}, 1.5) = \mathbf{1.500}$$

$$F1 = \mathbf{1.000}$$

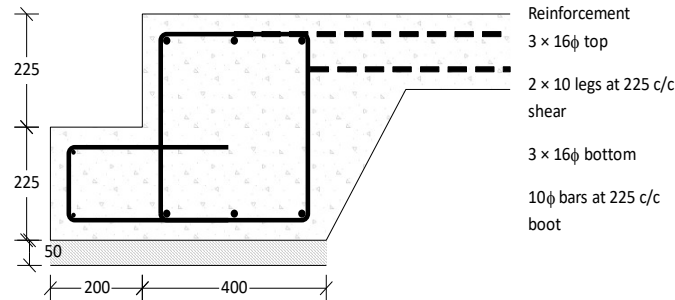
$$F2 = \mathbf{1.000}$$

$$\delta_{allow_slab} = \min(\delta_{basic_slab} \times K_{s_slab} \times F1 \times F2, 40 \times K_{b,defl_slab}) = \mathbf{60.000}$$

$$\delta_{actual_slab} = l_{eff_slab} / d_{avg_b_slab} = \mathbf{32.905}$$

PASS - Actual span to depth ratio is within the allowable limit

Edge beam 1 details



Section through beam

Depth of beam

Width of beam

Diameter of depression modified for hardcore

Angle of chamfer

Depth of boot

Width of boot

Depth of hardcore fill

Effective width of beam bearing

Nominal cover to edge beam top reinforcement

Nominal cover to edge beam bottom reinforcement

Top reinforcement provided

$$h_{edge} = \mathbf{450 \text{ mm}}$$

$$b_{edge} = \mathbf{400 \text{ mm}}$$

$$\phi_{dep_edge} = \phi_{dep_basic} - h_{fill_edge_b1} = \mathbf{3450 \text{ mm}}$$

$$\alpha_{edge} = \mathbf{60 \text{ deg}}$$

$$h_{boot_edge} = \mathbf{225 \text{ mm}}$$

$$b_{boot_edge} = \mathbf{200 \text{ mm}}$$

$$h_{fill_edge} = \mathbf{50 \text{ mm}}$$

$$b_{bearing_edge} = b_{edge} + b_{boot_edge} + (h_{edge} - h_{slab}) / \tan(\alpha_{edge}) = \mathbf{773 \text{ mm}}$$

$$c_{nom_edge_t} = \mathbf{35 \text{ mm}}$$

$$c_{nom_edge_b} = \mathbf{35 \text{ mm}}$$

$$\mathbf{3 \times 16\phi}$$

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Area of top reinforcement provided

$$A_{s,prov_t_edge} = 603 \text{ mm}^2$$

Effective depth of top reinforcement

$$d_{t_edge} = h_{edge} - C_{nom_edge_t} - \phi_{v_edge} - \phi_{t_edge} / 2 = 397 \text{ mm}$$

Bottom reinforcement provided

$$3 \times 16\phi$$

Area of bottom reinforcement provided

$$A_{s,prov_b_edge} = 603 \text{ mm}^2$$

Effective depth of bottom reinforcement

$$d_{b_edge} = h_{edge} - C_{nom_edge_b} - \phi_{v_edge} - \phi_{b_edge} / 2 = 397 \text{ mm}$$

Shear reinforcement provided

$$2 \times 10 \text{ legs at } 225 \text{ c/c}$$

Area of shear reinforcement provided

$$A_{sv,prov_edge} = 698 \text{ mm}^2/\text{m}$$

Boot reinforcement provided

$$10\phi \text{ bars at } 225 \text{ c/c}$$

Area of boot reinforcement provided

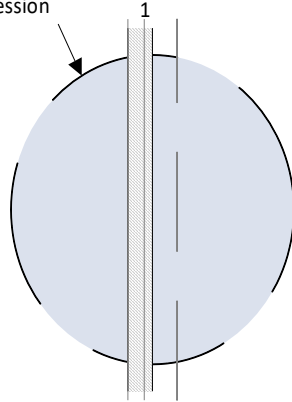
$$A_{s,prov_boot_edge} = 349 \text{ mm}^2/\text{m}$$

Effective depth of boot reinforcement

$$d_{boot_edge} = h_{boot_edge} - C_{nom_edge_b} - \phi_{boot_edge} / 2 = 185 \text{ mm}$$

Loading

3450 dia. of depression



Slab UDL's

Permanent UDL 2.0 kN/m²

Variable UDL 3.0 kN/m²

Additional loading

1 - Longitudinal: F_{Gk} 16.7 kN/m F_{Qk} 12.3 kN/m

Edge beam loading details

	Load type	Permanent	Variable	Width x	Width y	Centroid
1	Longitudinal	16.7 kN/m	12.3 kN/m	300 mm	-	150 mm

Edge beam design check

Edge beam self weight

$$W_{edge} = \rho_{conc} \times (h_{edge} \times b_{edge} + (h_{edge} - h_{slab_b1})^2 / (2 \times \tan(\alpha_{edge})) + h_{slab_b1} \times (h_{edge} - h_{slab_b1}) / \tan(\alpha_{edge}) + h_{boot_edge} \times b_{boot_edge}) = 6.8 \text{ kN/m}$$

Edge beam fill self weight

$$W_{fill_edge} = \rho_{fill} \times h_{fill_edge} = 1.0 \text{ kN/m}^2$$

Total uniform load at formation level

$$F_{edge} = W_{edge} / b_{bearing_edge} + W_{fill_edge} + W_{G_slab} + W_{Q_slab} = 14.8 \text{ kN/m}^2$$

Centroid of longitudinal and equivalent line loads from outside face of raft

Load $\times$ distance of load 1

$$M_{centroid_1_edge} = W_{ult_1_edge} \times X_{1_edge} = 6.1 \text{ kN}$$

Total load $\times$ distance

$$M_{centroid_total_edge} = M_{centroid_1_edge} = 6.1 \text{ kN}$$

Total ultimate long. and equiv. line loads

$$W_{equiv_total_edge} = W_{ult_1_edge} = 40.9 \text{ kN/m}$$

Centroid of loads

$$X_{bar_edge} = M_{centroid_total_edge} / W_{equiv_total_edge} = 150.0 \text{ mm}$$

Bearing pressure

Total unfactored longitudinal and eff. line loads

$$W_{equiv_total_sls_edge} = F_{Gk, line_1_edge} + F_{Qk, line_1_edge} = 29.0 \text{ kN/m}$$

Allowable bearing width

$$b_{brg_allow_edge} = 2 \times X_{bar_edge} + 2 \times h_{fill_slab_b1} / \tan(\alpha_{fill}) = 473 \text{ mm}$$

Bearing pressure due to line and point loads

$$F_{brg_edge} = W_{equiv_total_sls_edge} / b_{brg_allow_edge} = 61.2 \text{ kN/m}^2$$

Total applied bearing pressure

$$F_{brg_total_edge} = F_{brg_edge} + F_{edge} = 76.0 \text{ kN/m}^2$$

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$F_{brg_total} > q_{allow}$ - the slab is required to resist some of the beam loading

Assume moment due to load/reaction is resisted by the slab

Bearing width required	$b_{req_edge} = W_{equiv_total_sls_edge} / (q_{allow} - F_{edge}) = 822 \text{ mm}$
Effective bearing width at u/s of slab	$b_{req_eff_edge} = b_{req_edge} - 2 \times h_{fill_slab} / \tan(\alpha_{fill}) = 649 \text{ mm}$
Load/reaction eccentricity	$e_{slab_edge} = b_{req_eff_edge} / 2 - x_{bar_edge} = 174 \text{ mm}$
Ultimate moment to be resisted by slab	$M_{ecc_slab_edge} = W_{equiv_total_edge} \times e_{slab_edge} = 7.1 \text{ kNm/m}$
Total moment to be resisted by slab top steel	$M_{Ed_slab_total_edge} = M_{ecc_slab_edge} + M_{neg_total_slab} = 11.7 \text{ kNm/m}$
	$K_{slab_t_edge} = M_{Ed_slab_total_edge} / (d_{min_t_slab}^2 \times f_{ck}) = 0.036$
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = 0.207$
Lever arm	$Z_{slab_t_edge} = \min(0.5 \times d_{min_t_slab} \times [1 + (1 - 2 \times K_{slab_t_edge} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{min_t_slab}) = 95 \text{ mm}$
Minimum area of reinforcement required	$A_{s,min_slab_t_edge} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{min_t_slab} = 157 \text{ mm}^2/\text{m}$
Area of tension reinforcement required	$A_{s,req_slab_t_edge} = M_{Ed_slab_total_edge} / (f_{yd} \times Z_{slab_t_edge}) = 283 \text{ mm}^2/\text{m}$

PASS - Area of reinforcement provided in top to span local depression is adequate

Edge beam bending check

Divider for moments due to UDL's	$\beta_{udl} = 10.0$
Divider for moments due to point loads	$\beta_{pl} = 6.0$

Applied forces

Span of beam	$l_{edge} = \phi_{dep_edge} + d_{t_edge} = 3847 \text{ mm}$
Ultimate self weight UDL	$W_{ult_edge} = 1.35 \times W_{edge} = 9.2 \text{ kN/m}$
Approximate ultimate slab self weight UDL	$W_{ult_slab_edge} = \max(0 \text{ kN/m}, 1.35 \times W_{slab} \times (\phi_{dep_edge} / 2 \times 3/4 - (b_{edge} + (h_{edge} - h_{slab_b1}) / \tan(\alpha_{edge})))) = 3.6 \text{ kN/m}$
Self weight and slab bending moment	$M_{sw_edge} = (W_{ult_edge} + W_{ult_slab_edge}) \times l_{edge}^2 / \beta_{udl} = 18.8 \text{ kNm}$
Self weight and slab shear force	$V_{sw_edge} = (W_{ult_edge} + W_{ult_slab_edge}) \times l_{edge} / 2 = 24.5 \text{ kN}$

Moments due to applied uniformly distributed loads

Ultimate udl (approx)	$W_{ult_udl_edge} = (1.35 \times W_{G_slab} + 1.50 \times W_{Q_slab}) \times \phi_{dep_edge} / 2 \times 3/4 = 9.3 \text{ kN/m}$
Bending moment	$M_{udl_edge} = W_{ult_udl_edge} \times l_{edge}^2 / \beta_{udl} = 13.8 \text{ kNm}$
Shear force	$V_{udl_edge} = W_{ult_udl_edge} \times l_{edge} / 2 = 17.9 \text{ kN}$

Moment and shear load 1

Bending moment	$M_{1_edge} = W_{ult_1_edge} \times l_{edge}^2 / \beta_{udl} = 60.6 \text{ kNm}$
Shear force	$V_{1_edge} = W_{ult_1_edge} \times l_{edge} / 2 = 78.7 \text{ kN}$

Resultant moments and shears

Total bending moment (sagging and hogging)	$M_{total_edge} = M_{udl_edge} + M_{sw_edge} + M_{1_edge} = 93.2 \text{ kNm}$
Maximum shear force	$V_{total_edge} = V_{udl_edge} + V_{sw_edge} + V_{1_edge} = 121.1 \text{ kN}$

Reinforcement required in top of beam

Width of section in compression zone	$b_{t_comp_edge} = b_{edge} + b_{boot_edge} = 600 \text{ mm}$
	$K_{t_edge} = M_{total_edge} / (b_{t_comp_edge} \times d_{t_edge}^2 \times f_{ck}) = 0.031$
	$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = 0.207$

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Lever arm

$$Z_{t_edge} = \min(0.5 \times d_{t_edge} \times [1 + (1 - 2 \times K_{t_edge} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{t_edge}) = \mathbf{377 \text{ mm}}$$

Minimum area of reinforcement required

$$A_{s,min_t_edge} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b_{edge} \times d_{t_edge} = \mathbf{250 \text{ mm}^2}$$

Area of tension reinforcement required

$$A_{s,req_t_edge} = M_{total_edge} / (f_{yd} \times Z_{t_edge}) = \mathbf{568 \text{ mm}^2}$$

PASS - Area of reinforcement provided exceeds area of reinforcement required

Reinforcement required in bottom of beam

Effective flange outstand

$$b_{eff,1_edge} = 0.1 \times l_{edge} = \mathbf{385 \text{ mm}}$$

Effective flange width

$$b_{eff_edge} = b_{eff,1_edge} + b_{edge} + (h_{edge} - h_{slab_b1}) / \tan(\alpha_{edge}) = \mathbf{958 \text{ mm}}$$

$$K_{b_edge} = M_{total_edge} / (b_{eff_edge} \times d_{b_edge}^2 \times f_{ck}) = \mathbf{0.019}$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$$

Lever arm

$$Z_{b_edge} = \min(0.5 \times d_{b_edge} \times [1 + (1 - 2 \times K_{b_edge} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{b_edge}) = \mathbf{377 \text{ mm}}$$

Depth of neutral axis

$$x = 2 \times (d_{b_edge} - Z_{b_edge}) / \lambda = \mathbf{50 \text{ mm}}$$

$\lambda x \leq h_{slab}$ - Compression block lies wholly within the depth of slab

Minimum area of reinforcement required

$$A_{s,min_b_edge} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b_{edge} \times d_{b_edge} = \mathbf{250 \text{ mm}^2}$$

Area of tension reinforcement required

$$A_{s,req_b_edge} = M_{total_edge} / (f_{yd} \times Z_{b_edge}) = \mathbf{568 \text{ mm}^2}$$

PASS - Area of reinforcement provided in top to span local depression is adequate

Shear resistance with no shear reinforcement

Strength reduction factor

$$v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2) = \mathbf{0.523}$$

Max allowable design shear force (cl.6.2.2)

$$V_{Ed,max_edge} = 0.5 \times d_{t_edge} \times b_{edge} \times v_1 \times f_{cwd} = \mathbf{886.2 \text{ kN}}$$

PASS - Design shear force in slab is less than the maximum allowable shear force

Reinforcement ratio

$$\rho_{l_edge} = \min(A_{s,prov_t_edge} / (b_{edge} \times d_{t_edge}), 0.02) = \mathbf{0.00380}$$

k coefficient

$$k_{v_edge} = \min(1 + (200 \text{ mm} / d_{t_edge})^{0.5}, 2) = \mathbf{1.710}$$

C_{Rd} coefficient

$$C_{Rd,c_edge} = 0.18 / \gamma_c = \mathbf{0.120}$$

Minimum shear stress

$$v_{min_edge} = 0.035 \text{ N/mm}^2 \times k_{v_edge}^{3/2} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} = \mathbf{0.443 \text{ N/mm}^2}$$

Minimum design shear resistance

$$V_{Rd,c,min_edge} = v_{min_edge} \times d_{t_edge} \times b_{edge} = \mathbf{70.3 \text{ kN}}$$

Design shear resistance

$$V_{Rd,c_edge} = \max(C_{Rd,c_edge} \times k_{v_edge} \times 1 \text{ N/mm}^2 \times (100 \times \rho_{l_edge} \times f_{ck} / 1 \text{ N/mm}^2)^{1/3} \times d_{t_edge} \times b_{edge}, V_{Rd,c,min_edge}) = \mathbf{74.9 \text{ kN}}$$

Design shear reinforcement required

Angle of comp. shear strut for maximum shear

$$\theta_{max} = \mathbf{45 \text{ deg}}$$

Compression chord coefficient - cl.6.2.3(3)

$$\alpha_{cw} = \mathbf{1.00}$$

Minimum area of shear reinforcement - exp.9.5N

$$A_{sv,min_edge} = 0.08 \text{ N/mm}^2 \times b_{edge} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} = \mathbf{362 \text{ mm}^2/\text{m}}$$

Min lever arm in shear zone

$$Z_{min_edge} = \min(Z_{t_edge}, Z_{b_edge}) = \mathbf{377 \text{ mm}}$$

Maximum design shear resistance - exp.6.9

$$V_{Rd,max_edge} = \alpha_{cw} \times b_{edge} \times Z_{min_edge} \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max})) = \mathbf{841.9 \text{ kN}}$$

PASS - Design shear force at support is less than maximum design shear resistance

Design shear stress

$$v_{Ed_edge} = V_{total_edge} / (b_{edge} \times Z_{min_edge}) = \mathbf{0.803 \text{ N/mm}^2}$$

Angle of concrete compression strut - cl.6.2.3

$$\theta_{edge} = \min(\max(0.5 \times \text{Asin}(\min(2 \times v_{Ed_edge} / (\alpha_{cw} \times f_{cwd} \times v_1)), 1)), 21.8 \text{ deg}), 45 \text{ deg}) = \mathbf{21.8 \text{ deg}}$$

Area of shear reinforcement required - exp.6.8

$$A_{sv,des_edge} = v_{Ed_edge} \times b_{edge} / (f_{yd} \times \cot(\theta_{edge})) = \mathbf{296 \text{ mm}^2/\text{m}}$$

Area of shear reinforcement required

$$A_{sv,req_edge} = \max(A_{sv,min_edge}, A_{sv,des_edge}) = \mathbf{362 \text{ mm}^2/\text{m}}$$

Shear reinforcement provided

$$2 \times 10 \text{ legs at } 225 \text{ c/c}$$

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Area of shear reinforcement provided

$$A_{sv,prov_edge} = 698 \text{ mm}^2/\text{m}$$

PASS - Area of shear reinforcement provided exceeds minimum required

Boot design check

Effective cantilever span

$$l_{eff_boot_edge} = b_{boot_edge} + d_{boot_edge} / 2 = 293 \text{ mm}$$

Approximate ultimate bearing pressure

$$q_{approx_ult_edge} = 1.45 \times q_{allow} = 72.5 \text{ kN/m}^2$$

Cantilever moment

$$M_{boot_edge} = q_{approx_ult_edge} \times l_{eff_boot_edge}^2 / 2 = 3.1 \text{ kNm/m}$$

Shear force

$$V_{boot_edge} = q_{approx_ult_edge} \times l_{eff_boot_edge} = 21.2 \text{ kN/m}$$

$$K_{boot_edge} = M_{boot_edge} / (d_{boot_edge}^2 \times f_{ck}) = 0.003$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = 0.207$$

Lever arm

$$Z_{boot_edge} = \min(0.5 \times d_{boot_edge} \times [1 + (1 - 2 \times K_{boot_edge} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{boot_edge}) = 176 \text{ mm}$$

Minimum area of reinforcement required

$$A_{s,min_boot_edge} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{boot_edge} = 291 \text{ mm}^2/\text{m}$$

Area of tension reinforcement required

$$A_{s,req_boot_edge} = M_{boot_edge} / (f_{yd} \times Z_{boot_edge}) = 41 \text{ mm}^2/\text{m}$$

PASS - Area of reinforcement provided in boot exceeds area of reinforcement required

Shear resistance with no shear reinforcement

Strength reduction factor

$$v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2) = 0.523$$

Max allowable design shear force (cl.6.2.2)

$$V_{Ed,max_boot_edge} = 0.5 \times d_{boot_edge} \times v_1 \times f_{cwd} = 1032 \text{ kN/m}$$

PASS - Design shear force in boot is less than the maximum allowable shear force

Reinforcement ratio

$$\rho_{l_boot_edge} = \min(A_{s,prov_boot_edge} / d_{boot_edge}, 0.02) = 0.00189$$

k coefficient

$$k_{v_boot_edge} = \min(1 + (200 \text{ mm} / d_{boot_edge})^{0.5}, 2) = 2.000$$

$C_{Rd,c}$ coefficient

$$C_{Rd,c_boot_edge} = 0.18 / \gamma_c = 0.120$$

Minimum shear stress

$$v_{min_boot_edge} = 0.035 \text{ N/mm}^2 \times k_{v_boot_edge}^{3/2} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} = 0.560 \text{ N/mm}^2$$

Minimum design shear resistance

$$V_{Rd,c,min_boot_edge} = v_{min_boot_edge} \times d_{boot_edge} = 103.6 \text{ kN/m}$$

Design shear resistance

$$V_{Rd,c_boot_edge} = \max(C_{Rd,c_boot_edge} \times k_{v_boot_edge} \times 1 \text{ N/mm}^2 \times (100 \times \rho_{l_boot_edge} \times f_{ck} / 1 \text{ N/mm}^2)^{1/3} \times d_{boot_edge}, V_{Rd,c,min_boot_edge}) = 103.6 \text{ kN/m}$$

PASS - Design shear resistance exceeds design shear

Corner beam 1 details

Total uniform load at formation level

$$F_{corn} = W_{edge} / b_{bearing_edge} + W_{fill_edge} + W_{G_slab} + W_{Q_slab} = 14.8 \text{ kN/m}^2$$

PASS - Allowable bearing resistance exceeds applied bearing pressure

Applied forces

Span of beam

$$l_{corn} = \phi_{dep_corn} / \sqrt{2} + d_{t_edge} / 2 = 2638 \text{ mm}$$

Ultimate self weight UDL

$$W_{ult_corn} = 1.35 \times W_{edge} = 9.2 \text{ kN/m}$$

Approximate ultimate slab self weight UDL

$$W_{ult_slab_corn} = \max(0 \text{ kN/m}, 1.35 \times W_{slab} \times (\phi_{dep_corn} / (\sqrt{2} \times 2) - (b_{edge} + (h_{edge} - h_{slab_b1}) / \tan(\alpha_{edge})))) = 3.2 \text{ kN/m}$$

Self weight and slab bending moment

$$M_{sw_corn} = (W_{ult_corn} + W_{ult_slab_corn}) \times l_{corn}^2 / 2 = 43.0 \text{ kNm}$$

Self weight and slab shear force

$$V_{sw_corn} = (W_{ult_corn} + W_{ult_slab_corn}) \times l_{corn} = 32.6 \text{ kN}$$

Moments due to applied uniformly distributed loads

Ultimate udl (approx)

$$W_{ult_udl_corn} = (1.35 \times W_{G_slab} + 1.50 \times W_{Q_slab}) \times \phi_{dep_corn} / \sqrt{2} = 17.6 \text{ kN/m}$$

Bending moment

$$M_{udl_corn} = W_{ult_udl_corn} \times l_{corn}^2 / 6 = 20.4 \text{ kNm}$$

Shear force

$$V_{udl_corn} = W_{ult_udl_corn} \times l_{corn} / 2 = 23.2 \text{ kN}$$

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Resultant moments and shears

Bending moment

$$M_{total_x_corn} = M_{udl_corn} + M_{sw_corn} = \mathbf{63.4 \text{ kNm}}$$

Shear force

$$V_{total_x_corn} = V_{udl_corn} + V_{sw_corn} = \mathbf{55.8 \text{ kN}}$$

Bending moment

$$M_{total_y_corn} = M_{udl_corn} + M_{sw_corn} = \mathbf{63.4 \text{ kNm}}$$

Shear force

$$V_{total_y_corn} = V_{udl_corn} + V_{sw_corn} = \mathbf{55.8 \text{ kN}}$$

Average bending moment

$$M_{total_corn} = (M_{total_x_corn} + M_{total_y_corn}) / 2 = \mathbf{63.4 \text{ kNm}}$$

Average shear force

$$V_{total_corn} = (V_{total_x_corn} + V_{total_y_corn}) / 2 = \mathbf{55.8 \text{ kN}}$$

Reinforcement required in top of beam

$$K_{t_corn} = M_{total_corn} / (b_{t_comp_edge} \times d_{t_edge}^2 \times f_{ck}) = \mathbf{0.021}$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$$

Lever arm

$$Z_{t_corn} = \min(0.5 \times d_{t_edge} \times [1 + (1 - 2 \times K_{t_corn} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{t_edge}) = \mathbf{377 \text{ mm}}$$

Minimum area of reinforcement required

$$A_{s,min_t_corn} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b_{edge} \times d_{t_edge} = \mathbf{250 \text{ mm}^2}$$

Area of tension reinforcement required

$$A_{s,req_t_corn} = M_{total_corn} / (f_{yd} \times Z_{t_corn}) = \mathbf{387 \text{ mm}^2}$$

PASS - Area of reinforcement provided exceeds area of reinforcement required

Shear resistance with no shear reinforcement

Strength reduction factor

$$v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2) = \mathbf{0.523}$$

Max allowable design shear force (cl.6.2.2)

$$V_{Ed,max_corn} = 0.5 \times d_{t_edge} \times b_{edge} \times v_1 \times f_{cwd} = \mathbf{886.2 \text{ kN}}$$

PASS - Design shear force in slab is less than the maximum allowable shear force

Reinforcement ratio

$$\rho_{l_corn} = \min(A_{s,prov_t_edge} / (b_{edge} \times d_{t_edge}), 0.02) = \mathbf{0.00380}$$

k coefficient

$$k_{v_corn} = \min(1 + (200 \text{ mm} / d_{t_edge})^{0.5}, 2) = \mathbf{1.710}$$

$C_{Rd,c}$ coefficient

$$C_{Rd,c_corn} = 0.18 / \gamma_c = \mathbf{0.120}$$

Minimum shear stress

$$v_{min_corn} = 0.035 \text{ N/mm}^2 \times k_{v_corn}^{3/2} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} = \mathbf{0.443 \text{ N/mm}^2}$$

Minimum design shear resistance

$$V_{Rd,c,min_corn} = v_{min_corn} \times d_{t_edge} \times b_{edge} = \mathbf{70.3 \text{ kN}}$$

Design shear resistance

$$V_{Rd,c_corn} = \max(C_{Rd,c_corn} \times k_{v_corn} \times 1 \text{ N/mm}^2 \times (100 \times \rho_{l_corn} \times f_{ck} / 1 \text{ N/mm}^2)^{1/3} \times d_{t_edge} \times b_{edge}, V_{Rd,c,min_corn}) = \mathbf{74.9 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear

Deflection control - Section 7.4

Reference reinforcement ratio

$$\rho_{0_t_corn} = (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / 1000 = \mathbf{0.00566}$$

Required tension reinforcement ratio

$$\rho_{t_corn} = A_{s,req_t_corn} / (b_{edge} \times d_{t_edge}) = \mathbf{0.00244}$$

Required compression reinforcement ratio

$$\rho'_{t_corn} = \mathbf{0}$$

Structural system factor - Table 7.4N

$$K_{b,defl_t_corn} = \mathbf{0.4}$$

Basic allowable span to depth ratio

$$\delta_{basic_t_corn} = K_{b,defl_t_corn} \times [11 + 1.5 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times \rho_{0_t_corn} / \rho_{t_corn} + 3.2 \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} \times (\rho_{0_t_corn} / \rho_{t_corn} - 1)^{1.5}] = \mathbf{23.304}$$

Reinforcement factor - exp.7.17

$$K_{s_t_corn} = \min(A_{s,prov_t_edge} / A_{s,req_t_corn} \times 500 \text{ N/mm}^2 / f_{yk}, 1.5) = \mathbf{1.500}$$

Flange width factor

$$F1 = \mathbf{1.000}$$

Long span supporting brittle partition factor

$$F2 = \mathbf{1.000}$$

Allowable span to depth ratio

$$\delta_{allow_t_corn} = \min(\delta_{basic_t_corn} \times K_{s_t_corn} \times F1 \times F2, 40 \times K_{b,defl_t_corn}) = \mathbf{16.000}$$

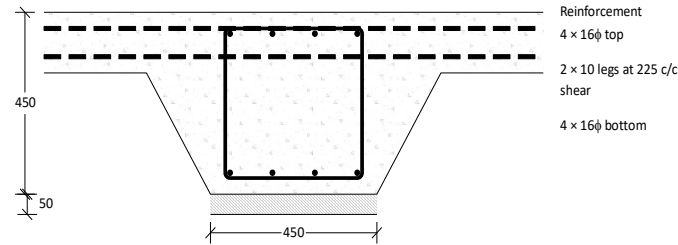
Actual span to depth ratio

$$\delta_{actual_t_corn} = l_{corn} / d_{t_edge} = \mathbf{6.645}$$

PASS - Actual span to depth ratio is within the allowable limit

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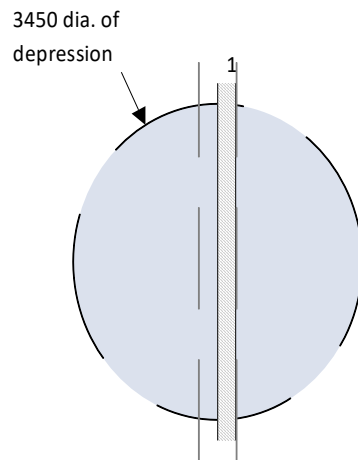
Internal beam 1 details



Section through beam

Depth of beam	$h_{int} = 450 \text{ mm}$
Width of beam	$b_{int} = 450 \text{ mm}$
Diameter of depression modified for hardcore	$\phi_{dep_int} = \phi_{dep_basic} - h_{fill_int_b1} = 3450 \text{ mm}$
Angle of chamfer	$\alpha_{int} = 60 \text{ deg}$
Depth of hardcore fill	$h_{fill_int} = 50 \text{ mm}$
Nominal cover to internal beam top reinforcement	$C_{nom_int_t} = 35 \text{ mm}$
Nom. cover to internal beam bottom reinforcement	$C_{nom_int_b} = 35 \text{ mm}$
Top reinforcement provided	$4 \times 16\phi$
Area of top reinforcement provided	$A_{s,prov_t_int} = 804 \text{ mm}^2$
Effective depth of top reinforcement	$d_{t_int} = h_{int} - C_{nom_int_t} - \phi_{v_int} - \phi_{t_int} / 2 = 397 \text{ mm}$
Bottom reinforcement provided	$4 \times 16\phi$
Area of bottom reinforcement provided	$A_{s,prov_b_int} = 804 \text{ mm}^2$
Effective depth of bottom reinforcement	$d_{b_int} = h_{int} - C_{nom_int_b} - \phi_{v_int} - \phi_{b_int} / 2 = 397 \text{ mm}$
Shear reinforcement provided	$2 \times 10 \text{ legs at } 225 \text{ c/c}$
Area of shear reinforcement provided	$A_{sv,prov_int} = 698 \text{ mm}^2/\text{m}$

Loading



Slab UDL's
Permanent UDL 2.0 kN/m^2
Variable UDL 3.0 kN/m^2

Additional loading
1 - Longitudinal: $F_{Gk} 14.8 \text{ kN/m}$ $F_{Qk} 8.5 \text{ kN/m}$

Internal beam loading details

	Load type	Permanent	Variable	Width x	Width y	Centroid
1	Longitudinal	14.8 kN/m	8.5 kN/m	215 mm	-	107 mm

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Internal beam design check

Int beam self weight

$$W_{int} = \rho_{conc} \times (h_{int} \times b_{int} + (h_{int} - h_{slab_b1})^2 / \tan(\alpha_{int}) + 2 \times h_{slab_b1} \times (h_{int} - h_{slab_b1}) / \tan(\alpha_{int})) = 7.5 \text{ kN/m}$$

Int beam fill self weight

$$W_{fill_int} = \rho_{fill} \times h_{fill_int} = 1.0 \text{ kN/m}^2$$

Total uniform load at formation level

$$F_{int} = W_{int} / (b_{int} + 2 \times (h_{int} - h_{slab_b1}) / \tan(\alpha_{int})) + W_{fill_int} + W_{G_slab} + W_{Q_slab} = 15.4 \text{ kN/m}^2$$

Equivalent and total beam line loads

Total factored long and effective line loads

$$W_{equiv_total_int} = W_{ult_1_int} = 32.7 \text{ kN/m}$$

Total unfactored long. and effective line loads

$$W_{equiv_total_sls_int} = F_{Gk, line_1_int} + F_{Qk, line_1_int} = 23.3 \text{ kN/m}$$

Centroid of loads from centreline of internal beam

Load $\times$ distance of load 1

$$M_{centroid_1_int} = W_{ult_1_int} \times X_{1_int} = 3.5 \text{ kN}$$

Total load $\times$ distance

$$M_{centroid_total_int} = M_{centroid_1_int} = 3.5 \text{ kN}$$

Centroid of loads

$$X_{bar_int} = M_{centroid_total_int} / W_{equiv_total_int} = 107.0 \text{ mm}$$

Moment due to eccentricity to be resisted by slab

$$M_{Ed_ecc_int} = W_{equiv_total_int} \times \text{abs}(X_{bar_int}) = 3.5 \text{ kNm/m}$$

Assume moment due to eccentricity is resisted equally by top steel of slab on one side and bottom steel of slab on other

Moment due to depression under slab

$$M_{Ed_neg_slab_total_int} = M_{neg_total_slab} + M_{Ed_ecc_int} / 2 = 6.3 \text{ kNm/m}$$

$$K_{slab_t_int} = M_{Ed_neg_slab_total_int} / (d_{min_t_slab}^2 \times f_{ck}) = 0.020$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = 0.207$$

Lever arm

$$Z_{slab_t_int} = \min(0.5 \times d_{min_t_slab} \times [1 + (1 - 2 \times K_{slab_t_int} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{min_t_slab}) = 95 \text{ mm}$$

Minimum area of reinforcement required

$$A_{s, min_slab_t_int} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{min_t_slab} = 157 \text{ mm}^2/\text{m}$$

Area of tension reinforcement required

$$A_{s, req_slab_t_int} = M_{Ed_neg_slab_total_int} / (f_{yd} \times Z_{slab_t_int}) = 152 \text{ mm}^2/\text{m}$$

PASS - Area of reinforcement provided in top to span local depression is adequate

Moment to be resisted in bot due to ecc.

$$M_{Ed_pos_slab_total_int} = M_{Ed_ecc_int} / 2 = 1.7 \text{ kNm/m}$$

$$K_{slab_b_int} = M_{Ed_pos_slab_total_int} / (d_{min_b_slab}^2 \times f_{ck}) = 0.005$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = 0.207$$

Lever arm

$$Z_{slab_b_int} = \min(0.5 \times d_{min_b_slab} \times [1 + (1 - 2 \times K_{slab_b_int} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{min_b_slab}) = 95 \text{ mm}$$

Minimum area of reinforcement required

$$A_{s, min_slab_b_int} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{min_b_slab} = 157 \text{ mm}^2/\text{m}$$

Area of tension reinforcement required

$$A_{s, req_slab_b_int} = M_{Ed_pos_slab_total_int} / (f_{yd} \times Z_{slab_b_int}) = 42 \text{ mm}^2/\text{m}$$

PASS - Area of reinforcement provided in top to span local depression is adequate

Bearing pressure

Allowable bearing width

$$b_{brg_allow_int} = \min(2 \times (b_{int} / 2 \times \tan(\alpha_{int}) + h_{int}) / (1 + \tan(\alpha_{int})), b_{int} + 2 \times (h_{int} - h_{slab_b1}) / \tan(\alpha_{int})) = 615 \text{ mm}$$

Bearing pressure due to line and point loads

$$F_{brg_int} = W_{equiv_total_sls_int} / b_{brg_allow_int} = 37.9 \text{ kN/m}^2$$

Total applied bearing pressure

$$F_{brg_total_int} = F_{brg_int} + F_{int} = 53.3 \text{ kN/m}^2$$

$F_{brg_total} > q_{allow}$ - the slab is required to resist some of the beam loading

Now consider slab to resist some of the beam load

Bearing width required

$$b_{brg_req_int} = W_{equiv_total_sls_int} / (q_{allow} - F_{int}) = 673 \text{ mm}$$



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Effective bearing width at u/s of slab	$b_{brg_req_eff_int} = b_{brg_req_int} - 2 \times h_{fill_int} / \tan(\alpha_{fill}) = 616 \text{ mm}$
Resultant ult upward pressure acting on u/s of slab	$F_{brg_res_int} = (q_{allow} - F_{int}) \times b_{brg_req_int} / b_{brg_req_eff_int} \times W_{equiv_total_int} / W_{equiv_total_sls_int} = 53.1 \text{ kN/m}^2$
Slab cantilever moment	$M_{Ed_cant_int} = F_{brg_res_int} \times ((b_{brg_req_int} - b_{int}) / 2)^2 / 2 = 0.3 \text{ kNm/m}$
Total moment to be resisted by slab	$M_{Ed_slab_total_int} = M_{Ed_cant_int} + M_{Ed_ecc_int} / 2 = 2.1 \text{ kNm/m}$ $K_{brg_b_int} = M_{Ed_slab_total_int} / (d_{min_b_slab}^2 \times f_{ck}) = 0.007$ $K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = 0.207$
Lever arm	$Z_{brg_b_int} = \min(0.5 \times d_{min_b_slab} \times [1 + (1 - 2 \times K_{brg_b_int} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{min_b_slab}) = 95 \text{ mm}$
Minimum area of reinforcement required	$A_{s,min_brg_b_int} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d_{min_b_slab} = 157 \text{ mm}^2/\text{m}$
Area of tension reinforcement required	$A_{s,req_brg_b_int} = M_{Ed_slab_total_int} / (f_{yd} \times Z_{brg_b_int}) = 50 \text{ mm}^2/\text{m}$
PASS - Area of reinforcement provided to distribute the load is adequate. The allowable bearing pressure will not be exceeded	
Internal beam bending check	
Divider for moments due to UDL's	$\beta_{udl} = 10.0$
Divider for moments due to point loads	$\beta_{pl} = 6.0$
Applied forces	
Span of beam	$l_{int} = \phi_{dep_int} + d_{t_int} = 3847 \text{ mm}$
Ultimate self weight UDL	$W_{ult_int} = 1.35 \times W_{int} = 10.1 \text{ kN/m}$
Approximate ultimate slab self weight UDL	$W_{ult_slab_int} = \max(0 \text{ kN/m}, 1.35 \times W_{slab} \times (\phi_{dep_int} \times 3/4 - (b_{int} + 2 \times (h_{int} - h_{slab_b1}) / \tan(\alpha_{int})))) = 8.9 \text{ kN/m}$
Self weight and slab bending moment	$M_{sw_int} = (W_{ult_int} + W_{ult_slab_int}) \times l_{int}^2 / \beta_{udl} = 28.1 \text{ kNm}$
Self weight and slab shear force	$V_{sw_int} = (W_{ult_int} + W_{ult_slab_int}) \times l_{int} / 2 = 36.6 \text{ kN}$
Moments due to applied uniformly distributed loads	
Ultimate udl (approx)	$W_{ult_udl_int} = (1.35 \times W_{G_slab} + 1.50 \times W_{Q_slab}) \times \phi_{dep_int} \times 3 / 4 = 18.6 \text{ kN/m}$
Bending moment	$M_{udl_int} = W_{ult_udl_int} \times l_{int}^2 / \beta_{udl} = 27.6 \text{ kNm}$
Shear force	$V_{udl_int} = W_{ult_udl_int} \times l_{int} / 2 = 35.8 \text{ kN}$
Moment and shear load 1	
Bending moment	$M_{1_int} = W_{ult_1_int} \times l_{int}^2 / \beta_{udl} = 48.4 \text{ kNm}$
Shear force	$V_{1_int} = W_{ult_1_int} \times l_{int} / 2 = 62.9 \text{ kN}$
Resultant moments and shears	
Total bending moment (sagging and hogging)	$M_{total_int} = M_{udl_int} + M_{sw_int} + M_{1_int} = 104.1 \text{ kNm}$
Maximum shear force	$V_{total_int} = V_{udl_int} + V_{sw_int} + V_{1_int} = 135.3 \text{ kN}$
Reinforcement required in top of beam	
	$K_{t_int} = M_{total_int} / (b_{int} \times d_{t_int}^2 \times f_{ck}) = 0.046$ $K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = 0.207$
Lever arm	$z_{t_int} = \min(0.5 \times d_{t_int} \times [1 + (1 - 2 \times K_{t_int} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{t_int}) = 377 \text{ mm}$
Minimum area of reinforcement required	$A_{s,min_t_int} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b_{int} \times d_{t_int} = 281 \text{ mm}^2$

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Area of tension reinforcement required

$$A_{s,req_t_int} = M_{total_int} / (f_{yd} \times Z_{t_int}) = \mathbf{635 \text{ mm}^2}$$

PASS - Area of reinforcement provided exceeds area of reinforcement required

Reinforcement required in bottom of beam

Effective flange outstand

$$b_{eff,1_int} = 0.1 \times l_{int} = \mathbf{385 \text{ mm}}$$

Effective flange width

$$b_{eff_int} = 2 \times b_{eff,1_int} + b_{int} + 2 \times (h_{int} - h_{slab_b1}) / \tan(\alpha_{int}) = \mathbf{1566 \text{ mm}}$$

$$K_{b_int} = M_{total_int} / (b_{eff_int} \times d_{b_int}^2 \times f_{ck}) = \mathbf{0.013}$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - k_1) / (2 \times k_2)) \times (\lambda \times (\delta - k_1) / (2 \times k_2)) = \mathbf{0.207}$$

Lever arm

$$Z_{b_int} = \min(0.5 \times d_{b_int} \times [1 + (1 - 2 \times K_{b_int} / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}], 0.95 \times d_{b_int}) = \mathbf{377 \text{ mm}}$$

Depth of neutral axis

$$x = 2 \times (d_{b_int} - Z_{b_int}) / \lambda = \mathbf{50 \text{ mm}}$$

$\lambda x \leq h_{slab}$ - Compression block lies wholly within the depth of slab

Minimum area of reinforcement required

$$A_{s,min_b_int} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times b_{int} \times d_{b_int} = \mathbf{281 \text{ mm}^2}$$

Area of tension reinforcement required

$$A_{s,req_b_int} = M_{total_int} / (f_{yd} \times Z_{b_int}) = \mathbf{635 \text{ mm}^2}$$

PASS - Area of reinforcement provided in top to span local depression is adequate

Shear resistance with no shear reinforcement

Strength reduction factor

$$v_1 = 0.6 \times (1 - f_{ck} / 250 \text{ N/mm}^2) = \mathbf{0.523}$$

Max allowable design shear force (cl.6.2.2)

$$V_{Ed,max_int} = 0.5 \times d_{t_int} \times b_{int} \times v_1 \times f_{cwd} = \mathbf{997.0 \text{ kN}}$$

PASS - Design shear force in slab is less than the maximum allowable shear force

Reinforcement ratio

$$\rho_{l_int} = \min(A_{s,prov_t_int} / (b_{int} \times d_{t_int}), 0.02) = \mathbf{0.00450}$$

k coefficient

$$k_{v_int} = \min(1 + (200 \text{ mm} / d_{t_int})^{0.5}, 2) = \mathbf{1.710}$$

C_{Rd} coefficient

$$C_{Rd,c_int} = 0.18 / \gamma_c = \mathbf{0.120}$$

Minimum shear stress

$$v_{min_int} = 0.035 \text{ N/mm}^2 \times k_{v_int}^{3/2} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} = \mathbf{0.443 \text{ N/mm}^2}$$

Minimum design shear resistance

$$V_{Rd,c,min_int} = v_{min_int} \times d_{t_int} \times b_{int} = \mathbf{79.1 \text{ kN}}$$

Design shear resistance

$$V_{Rd,c_int} = \max(C_{Rd,c_int} \times k_{v_int} \times 1 \text{ N/mm}^2 \times (100 \times \rho_{l_int} \times f_{ck} / 1 \text{ N/mm}^2)^{1/3} \times d_{t_int} \times b_{int}, V_{Rd,c,min_int}) = \mathbf{89.2 \text{ kN}}$$

Design shear reinforcement required

Angle of comp. shear strut for maximum shear

$$\theta_{max} = \mathbf{45 \text{ deg}}$$

Compression chord coefficient - cl.6.2.3(3)

$$\alpha_{cw} = \mathbf{1.00}$$

Minimum area of shear reinforcement - exp.9.5N

$$A_{sv,min_int} = 0.08 \text{ N/mm}^2 \times b_{int} \times (f_{ck} / 1 \text{ N/mm}^2)^{0.5} / f_{yk} = \mathbf{407 \text{ mm}^2/\text{m}}$$

Min lever arm in shear zone

$$Z_{min_int} = \min(Z_{t_int}, Z_{b_int}) = \mathbf{377 \text{ mm}}$$

Maximum design shear resistance - exp.6.9

$$V_{Rd,max_int} = \alpha_{cw} \times b_{int} \times Z_{min_int} \times v_1 \times f_{cwd} / (\cot(\theta_{max}) + \tan(\theta_{max})) = \mathbf{947.2 \text{ kN}}$$

PASS - Design shear force at support is less than maximum design shear resistance

Design shear stress

$$v_{Ed_int} = V_{total_int} / (b_{int} \times Z_{min_int}) = \mathbf{0.797 \text{ N/mm}^2}$$

Angle of concrete compression strut - cl.6.2.3

$$\theta_{int} = \min(\max(0.5 \times \text{Asin}(\min(2 \times v_{Ed_int} / (\alpha_{cw} \times f_{cwd} \times v_1)), 1)), 21.8 \text{ deg}), 45 \text{ deg}) = \mathbf{21.8 \text{ deg}}$$

Area of shear reinforcement required - exp.6.8

$$A_{sv,des_int} = v_{Ed_int} \times b_{int} / (f_{yd} \times \cot(\theta_{int})) = \mathbf{330 \text{ mm}^2/\text{m}}$$

Area of shear reinforcement required

$$A_{sv,req_int} = \max(A_{sv,min_int}, A_{sv,des_int}) = \mathbf{407 \text{ mm}^2/\text{m}}$$

Shear reinforcement provided

$$2 \times 10 \text{ legs at } 225 \text{ c/c}$$

Area of shear reinforcement provided

$$A_{sv,prov_int} = \mathbf{698 \text{ mm}^2/\text{m}}$$

PASS - Area of shear reinforcement provided exceeds minimum required



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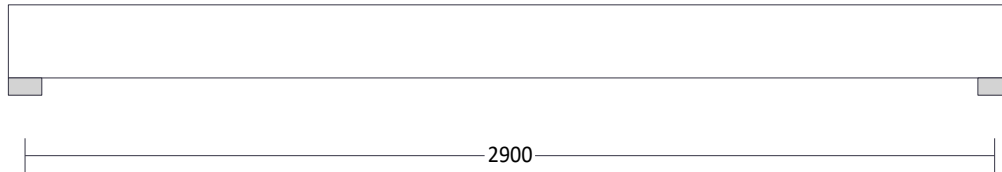
TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

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

Tedds calculation version 1.0.08

Joist details

Description 47 x 200 C24 timber joists
Joist spacing $s_{Joist} = 450$ mm



Forces input on Joist

Vertical permanent load on joist $F_{G_Joist} = 0.55$ kN/m²
Vertical imposed load on joist $F_{Q_Joist} = 1.50$ kN/m²

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G_Joist} \times s_{Joist} = 0.25$ kN/m
Vertical imposed load on joist $p_Q = F_{Q_Joist} \times s_{Joist} = 0.68$ kN/m

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	1.7	0.3	0.196	PASS
Bending stress	N/mm ²	16.2	4.7	0.289	PASS
Shear stress	N/mm ²	2.7	0.5	0.178	PASS
Beam stability check				0.289	PASS
Deflection	mm	11.6	5.0	0.428	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	0.25 kN/m at 0 m to 2.9 m
Member	Imposed	UDL	GlobalZ	0.68 kN/m at 0 m to 2.9 m



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Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.23631	
2	0	0	-0.23631	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.16238	
2	0	0	-0.16238	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	4.1
1.00G + 1.00Q (Service)	0	2.8

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	2.9	1	0	-2	0
		2	0	-2	0



JUST BEAMS
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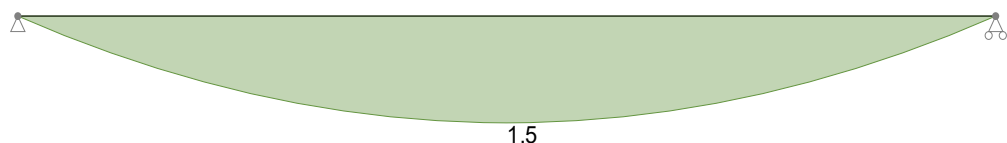
Date
06-11-2024

Load combination: 1.00G + 1.00Q (Service)

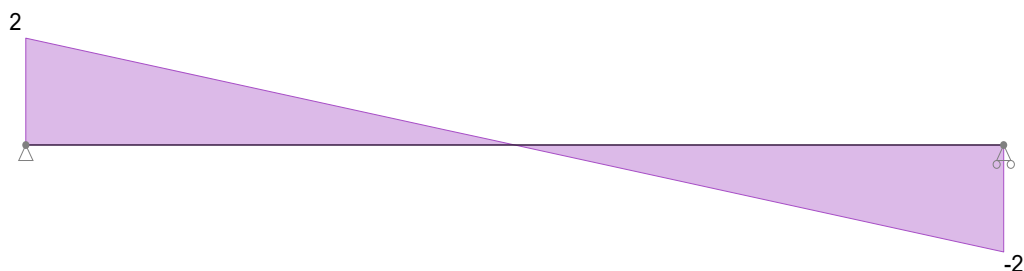
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.9	1	0	-1.4	0
		2	0	-1.4	0

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		0 (min)	
	1.45	0		1.5 (max)	
	2.9	-2 (max abs)		0 (min)	

Tedds calculation version 2.2.21

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 $b = 47$ mm

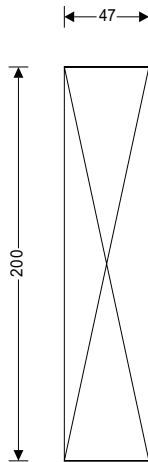
Depth of sections $h = 200$ mm

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



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47x200 timber section

Cross-sectional area, A , 9400 mm²

Section modulus, W_y , 313333.3 mm³

Section modulus, W_z , 73633 mm³

Second moment of area, I_y , 31333333 mm⁴

Second moment of area, I_z , 1730383 mm⁴

Radius of gyration, i_y , 57.7 mm

Radius of gyration, i_z , 13.6 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 $F_{c,y,90,d} = 2.028$ kN

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

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.332$ 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

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.196$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 2.028$ kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.483$ N/mm²

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

$\tau_{y,d} / f_{v,y,d} = 0.178$

PASS - Design shear strength exceeds design shear stress

Check design 1450 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 1.471$ kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 4.693$ N/mm²



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Design bending strength

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

$$\sigma_{m,y,d} / f_{m,y,d} = 0.289$$

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

$$k_{crit} = 1.000$$

Beam stability check - exp.6.33

$$\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}) = 0.289$$

PASS - Beam stability is acceptable

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

Check design 1450 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$$\delta_y = 2.8 \text{ mm}$$

Quasi-permanent variable load factor

$$\psi_2 = 0.3$$

Final deflection with creep

$$\delta_{y,Final} = \delta_y \times (1 + k_{def}) = 5 \text{ mm}$$

Allowable deflection

$$\delta_{y,Allowable} = L_{m1_s1} / 250 = 11.6 \text{ mm}$$

$$\delta_{y,Final} / \delta_{y,Allowable} = 0.428$$

PASS - Allowable deflection exceeds final deflection



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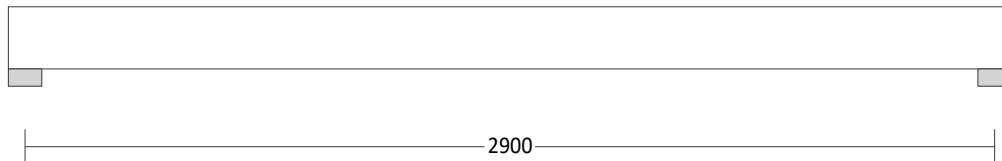
TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

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

Tedds calculation version 1.0.08

Joist details

Description 47 x 170 C24 timber joists
Joist spacing $s_{Joist} = 450$ mm



Forces input on Joist

Vertical permanent load on joist $F_{G_Joist} = 0.80$ kN/m²
Vertical imposed load on joist $F_{Q_Joist} = 0.75$ kN/m²

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G_Joist} \times s_{Joist} = 0.36$ kN/m
Vertical imposed load on joist $p_Q = F_{Q_Joist} \times s_{Joist} = 0.34$ kN/m

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	1.7	0.2	0.145	PASS
Bending stress	N/mm ²	16.2	4.8	0.296	PASS
Shear stress	N/mm ²	2.7	0.4	0.156	PASS
Beam stability check				0.296	PASS
Deflection	mm	11.6	6.0	0.519	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	0.36 kN/m at 0 m to 2.9 m
Member	Imposed	UDL	GlobalZ	0.34 kN/m at 0 m to 2.9 m



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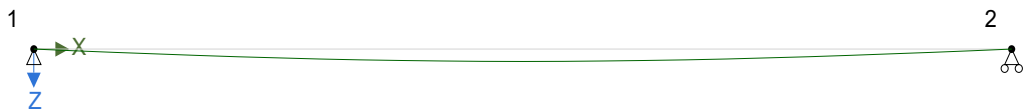
Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.28516	
2	0	0	-0.28516	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.20092	
2	0	0	-0.20092	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	3
1.00G + 1.00Q (Service)	0	2.1

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	2.9	1	0	-1.5	0
		2	0	-1.5	0



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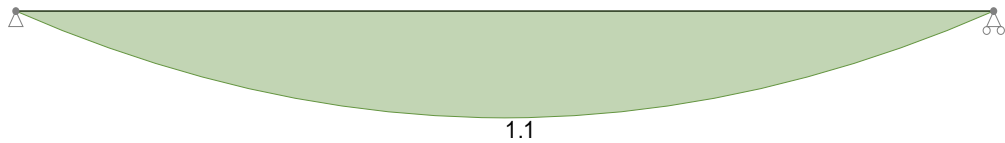
Date
06-11-2024

Load combination: 1.00G + 1.00Q (Service)

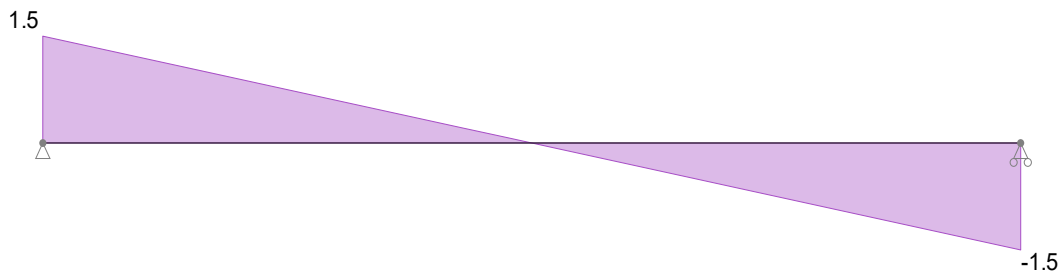
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.9	1	0	-1.1	0
		2	0	-1.1	0

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	1.5		0 (min)	
	1.45	0		1.1 (max)	
	2.9	-1.5 (max abs)		0 (min)	

Tedds calculation version 2.2.21

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

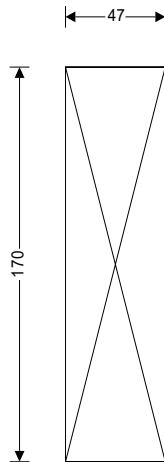
Timber section details

Number of timber sections in member $N = 1$
Breadth of sections $b = 47$ mm
Depth of sections $h = 170$ mm
Timber strength class - EN 338:2016 Table 1 **C24**



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47x170 timber section

Cross-sectional area, A , 7990 mm²
Section modulus, W_y , 226383.3 mm³
Section modulus, W_z , 62588 mm³
Second moment of area, I_y , 19242583 mm⁴
Second moment of area, I_z , 1470826 mm⁴
Radius of gyration, i_y , 49.1 mm
Radius of gyration, i_z , 13.6 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 $F_{c,y,90,d} = 1.503$ kN
Effective contact length $L_{b,ef} = L_b + \min(L_b, 30 \text{ mm}) = 130$ mm
Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.246$ 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
 $\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.145$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 1.503$ kN
Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.421$ N/mm²
Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.708$ N/mm²
 $\tau_{y,d} / f_{v,y,d} = 0.156$

PASS - Design shear strength exceeds design shear stress

Check design 1450 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 1.09$ kNm
Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 4.814$ N/mm²



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Design bending strength

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

$$\sigma_{m,y,d} / f_{m,y,d} = 0.296$$

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

$$k_{crit} = 1.000$$

Beam stability check - exp.6.33

$$\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}) = 0.296$$

PASS - Beam stability is acceptable

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

Check design 1450 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$$\delta_y = 3.3 \text{ mm}$$

Quasi-permanent variable load factor

$$\psi_2 = 0.3$$

Final deflection with creep

$$\delta_{y,Final} = \delta_y \times (1 + k_{def}) = 6 \text{ mm}$$

Allowable deflection

$$\delta_{y,Allowable} = L_{m1_s1} / 250 = 11.6 \text{ mm}$$

$$\delta_{y,Final} / \delta_{y,Allowable} = 0.519$$

PASS - Allowable deflection exceeds final deflection



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1.Calculation Details [Calculation 1]

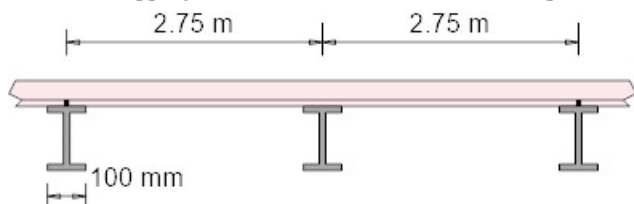
Design Standard	Eurocode	Locale	UK
Project Name	Maesy Coed Guest House	Prepared By	ZR
Location	Merthyr Tydfil, CF47 8UY	Company Name	Just Beams
Client Name		Date	06-11-2024
Client Company		Slab Reference	First floor
Reference		Checked By	ZR
Comments			

2. Overall Summary [SECTION ADEQUATE]

ULS Checks	Construction Stage	PASS	Max Unity Factor = 0.33
	Normal Stage	N/A	
	Fire Limit State	N/A	
SLS Checks		SATISFACTORY	Max Unity Factor = 0.40

3.General Arrangement

Deck Profile Span Type	Double	Equal Spans?	Yes
Side 1 Span Length	2.75 m	Side 2 Span Length	2.75 m
Concrete Slab Span Type	Internal	Support Width	100 mm
Deck Propping	No Props		



4.Loading Details

Concrete weight (wet)	2.93 kN/m ²	Concrete weight (dry)	2.93 kN/m ²
Decking self weight	0.15 kN/m ²	Imposed (occupancy)	5.00 kN/m ²
Reinforcement self weight	0.07 kN/m ²	Ceilings / services	0.50 kN/m ²
Construction load	0.75 kN/m ²	Finishes	0.00 kN/m ²
Construction load patch	0.75 kN/m ²	Partitions	1.00 kN/m ²
		Screed self weight	0.00 kN/m ²

• Parallel Loading

Not applied

• **Perpendicular Loading**

Not applied

• **Point Loading**

Not applied

5.Profile

• **Profile Details**

Profile Type	MetFloor® 80	Nominal Sheet Thickness	1.20 mm
Profile Yield	350 N/mm²	Effective Sheet Thickness	1.14 mm
Permanent Shuttering?	Yes		

• **Section Properties**

*** Note - 1: All values of inertia are expressed in steel units

*** Note - 2: Average inertia is used for deflection calculations for the composite stage

*** Note - 3: Cracked dynamic inertia is used for natural frequency calculation

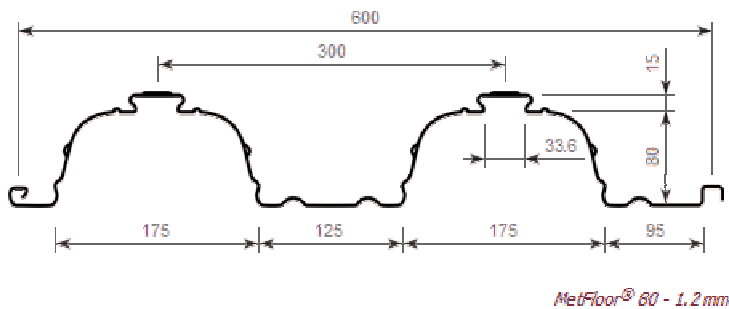
DECK PROFILE

Sagging Inertia, I_y	206.27 cm ⁴ /m	Cross section area	1833.00 mm ² /m
Hogging Inertia, I_y	184.15 cm ⁴ /m	Effective cross section	1648.90 mm ² /m

COMPOSITE

Inertia, I_y - Uncracked	0.00 cm ⁴ /m	Inertia, I_y - Cracked	0.00 cm ⁴ /m
Average inertia	0.00 cm ⁴ /m	Cracked inertia (dynamic)	0.00 cm ⁴ /m
Shear bond coefficient, M_r	140.90	Shear bond coefficient, K_r	0.122700
Concrete volume	0.12 m ³ /m/m		

• **Cross Section Details**



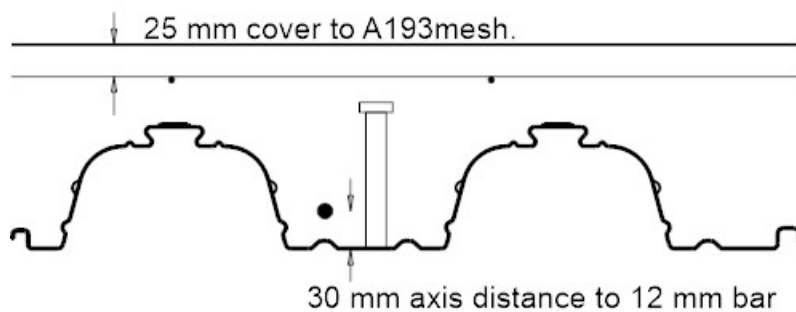
6.Slab

• Slab Details

Concrete Grade	C35/45	Slab Depth	160 mm
Concrete Type	Normal Weight	Modular Ratio	10
Wet Density	2550 kg/m ³	Dry Density	2450 kg/m ³
Mesh Type	A193	Mesh Cover	25
Mesh Layer	Single	Mesh Yield Stress	500 N/mm ²
Bar Diameter	12 mm	Bar Yield Stress	500 N/mm ²
		Axis Distance	30 mm
Account for End Anchorage	No	Number of Studs per Rib	N/A
Stud Diameter	N/A		
Screed Depth	50 mm	Screed Density	2400 kg/m ³

• Slab Drawing

MetFloor® 80 - 1.2 mm.



7.Analysis

• Fire Parameters

Design Method:	Bar	Fire Resistance Period:	60 min
----------------	-----	-------------------------	--------

• Deflection Limits

Construction Stage:	Span/180 Or 20
	Span/130 Or 30
Imposed Loads:	Span/350 Or 20
Total Loads:	Span/250 Or 20

• Partial Load Factors

Permanent	1.35	Variable	1.5
Superimposed permanent	N/A	Fire:	N/A

• Other

Longitudinal Shear Method	m & k
Natural frequency limit	5 Hz
Imposed Load Category	B - Office [Psi_0=0.7, Psi_1=0.5]

8. Construction Stage

• Effective Span of Deck

Effective span L_e , is the smaller of

- | | |
|----------------------------|-----------------------|
| 1) c/c of supports | 2.75 m |
| 2) clear span + deck depth | $2.65 + 80.00 / 1000$ |
| | 2.73 m |

Therefore $L_e = 2.73$ m

• Actions (Unfactored)

Permanent:

Self weight of decking 0.15 kN/m²

Reinforcement 0.07 kN/m²

Variable:

Self weight of concrete slab (wet) 2.93 kN/m²

Additional slab weight due to ponding 0.00 kN/m²

Construction load 0.75 kN/m²

Construction load patch 0.75 kN/m²

• Combination of Actions

Note: The load combination for the ULS verification is as defined in EN 1990 § 6.4.3.2 (6.10).

Note: The load combination for the SLS verification is the characteristic load combination as defined in EN 1990 § 6.5.3.

• Shear Resistance Check (BS EN 1993-1-3 Clause 6.1.5 and 6.1.7.3)

Applied shear 11.33 kN/m

Web shear resistance, P_v 103.05 kN/m

UNITY FACTOR $0.11 \leq 1.00$

PASS

Applied reaction 18.28 kN/m

Web crushing resistance, P_w 54.71 kN/m

UNITY FACTOR $0.33 \leq 1.00$

PASS

• Bending Resistance Check

*** Note: A 100% redistribution of hogging moment is taken based on equilibrium of the continuous decking as the sagging moment does not exceed the design resistance

Sagging:

Max applied moment 5.11 kNm/m

Moment resistance 19.10 kNm/m

UNITY FACTOR $0.27 \leq 1.00$

PASS

Hogging:

Applied moment 0.00 kNm/m

Moment resistance 15.19 kNm/m

UNITY FACTOR $0.00 \leq 1.00$

PASS

• Combined Effects

Bending and Web Crushing (BS EN 1993-1-3 Clause 6.1.11)

*** Note: A 100.00% redistribution of hogging moment is taken based on equilibrium of the continuous decking as the sagging moment does not exceed the design resistance

Unity Factor is the worst case of

$$1. \text{ Maximum hogging} = (9.83 / 54.71 + 0.00 / 15.19) / 1.42$$

$$= 0.13$$

$$2. \text{ Maximum reaction} = (18.28 / 54.71 + 0.00 / 15.19) / 1.42$$

$$= 0.24$$

$$\text{UNITY FACTOR} \quad 0.24 \leq 1.00$$

PASS

Bending and Shear (BS EN 1993-1-3 Clause 6.1.10)

*** Note: Low shear - This check is not required

Support Interaction Check at Serviceability Limit State (BS EN 1993-1-3 Clause 7.2)

Unity Factor is the worst case of

$$1. \text{ Maximum hogging} = (14.91 / 54.71 + 3.63 / 15.19) / (0.9 * 1.42)$$

$$= 0.40$$

$$2. \text{ Maximum reaction} = (14.91 / 54.71 + 3.63 / 15.19) / (0.9 * 1.42)$$

$$= 0.40$$

$$\text{UNITY FACTOR} \quad 0.40 \leq 1.00$$

PASS

• Deflection Check

Allowable deflection is the lesser of

1) Effective span / deflection limit without ponding	15.17 mm
2) Deflection limit without ponding, absolute maximum value	20.00 mm
3) Slab depth / 10	16.00 mm

$$\text{Max self weight deflection} = 2.42 \text{ mm} \leq 15.17 \text{ mm (UF=0.16)}$$

SATISFACTORY

Project MAES Y COED GUEST HOUSE, MERTHYR TYDFIL, CF47 8UY						Status BUILDERS REVIEW											
Date 28-05-2024		Job no. 65		Section -		Sheet no. 1										Rev R0	
By Z.RAHMAN																	
Checked -																	
Rev R0		Date 28-05-2024		Details BAR BENDING SCHEDULE					Tel +44(0) 7769 316378								
Fax -																	
Part																	
BEAMS AND SLAB MESH																	
Ref:	Orig. by		Date 28-05-2024		Ref. No.		Sheet No. 1 of 1		Bar Bending Schedule								
	Member	Mark	Type	Quantity	Length (mm)	Total Length (m)	Shape	A (mm)	B (mm)	C (mm)	D (mm)	E (mm)	F (mm)	Unit Mass (kg/m)	Total Mass (Kg)		
Beam 1	2	H16	12	1350	16	11	1005	360						1.578	25.57		
	L3	H12	4	1375	6	11	1245	135						0.888	4.88		
	R3	H12	4	1175	5	11	1045	135						0.888	4.17		
	5	H16	6	5400	32	0	5400							1.578	51.14		
	6	H12	2	5400	11	0	5400							0.888	9.59		
Beam 2	L2	H16	6	1600	10	11	1255	360						1.578	15.15		
	L3	H12	4	1225	5	11	1095	135						0.888	4.35		
	R2	H16	6	1400	8	11	1055	360						1.578	13.26		
	R3	H12	4	1025	4	11	895	135						0.888	3.64		
	7	H16	6	2350	14	0	2350							1.578	22.25		
Beam 3	8	H12	2	2350	5	0	2350							0.888	4.17		
	L2	H16	6	1750	11	11	1405	360						1.578	16.57		
	R2	H16	6	1400	8	11	1055	360						1.578	13.26		
Beam 4	11	H16	6	4950	30	0	4950							1.578	46.88		
	L2	H16	6	1400	8	11	1055	360						1.578	13.26		
	L3	H12	4	1025	4	11	895	135						0.888	3.64		
	R2	H16	6	1400	8	11	1055	360						1.578	13.26		
	R3	H12	4	1025	4	11	895	135						0.888	3.64		
	12	H16	6	1300	8	0	4300							1.578	12.31		
Beam 5	13	H12	2	4300	9	0	4300							0.888	7.64		
	L2	H16	6	1350	8	11	1005	360						1.578	12.78		
	L3	H12	4	1375	6	11	1245	135						0.888	4.88		
	R2	H16	6	1400	8	11	1055	360						1.578	13.26		
	R3	H12	4	1025	4	11	895	135						0.888	3.64		
	15	H16	24	4725	113	0	4725							1.578	178.98		
	14	H16	18	1300	23	0	1300							1.578	36.93		
Beam 6	16	H12	6	1000	6	0	4725							0.888	5.33		
	17	H12	8	4725	38	0	1000							0.888	33.56		
	2	H16	12	1350	16	11	1005	360						1.578	25.57		
	3	H12	8	1175	9	11	1045	135						0.888	8.35		
Beam 7	18	H16	6	4900	29	0	4900							1.578	46.40		
	19	H12	2	4900	10	0	4900							0.888	8.70		
	2	H16	12	1350	16	11	1005	360						1.578	25.57		
Beam 8	21	H16	6	4200	25	0	4200							1.578	39.77		
	3	H12	8	1000	8	11	870	135						0.888	7.10		
	4	H16	12	1175	14	11	835	360						1.578	22.25		
	2	H16	36	1350	49	11	1005	360						1.578	76.71		
Beam 9	22	H16	12	3500	42	0	3500							1.578	66.29		
	10	H16	12	1575	19	11	1235	360						1.578	29.83		
	2	H16	24	1350	32	11	1005	360						1.578	51.14		
Beam 10	23	H16	12	5000	60	0	5000							1.578	94.70		
	Links																
Links Beam 1	1	H10	24	2550	61	64	565	155	730	380	330	130		0.617	37.73		
Links	1	H10	140	2150	301	64	365	155	530	380	330	130		0.617	185.57		
Links	9	H10	142	1700	241	51	380	380	130	130				0.617	148.83		
concrete slab																	
Slab	20	H10	110	1575	173	0	1450							0.617	106.81		
	19 No. of 4.8m x 2.4m A252 mesh required														864.50		
Screed	7 No. of 4.8m x 2.4m A98 mesh required														124.60		
Total Steel Mass (kg)														2620			



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28 HARROW ROAD, NEWPORT

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Section Retaining Wall				Sheet no./rev. 01/ 1	
Calc. by ZR	Date 05-12-2024	Chk'd by ZR	Date 05-12-2024	App'd by ZR	Date 05-12-2024

RETAINING WALL ANALYSIS

In accordance with EN1997-1:2004 incorporating Corrigendum dated February 2009 and the UK National Annex incorporating Corrigendum No.1

Tedds calculation version 2.9.22

Retaining wall details

Stem type	Cantilever
Stem height	$h_{\text{stem}} = 3250$ mm
Stem thickness	$t_{\text{stem}} = 300$ mm
Angle to rear face of stem	$\alpha = 90$ deg
Stem density	$\gamma_{\text{stem}} = 25$ kN/m ³
Toe length	$l_{\text{toe}} = 1200$ mm
Heel length	$l_{\text{heel}} = 500$ mm
Base thickness	$t_{\text{base}} = 250$ mm
Key position	$p_{\text{key}} = 900$ mm
Key depth	$d_{\text{key}} = 450$ mm
Key thickness	$t_{\text{key}} = 600$ mm
Base density	$\gamma_{\text{base}} = 25$ kN/m ³
Height of retained soil	$h_{\text{ret}} = 3000$ mm
Angle of soil surface	$\beta = 0$ deg
Depth of cover	$d_{\text{cover}} = 200$ mm

Retained soil properties

Soil type	Very dense gravel
Moist density	$\gamma_{\text{mr}} = 18$ kN/m ³
Saturated density	$\gamma_{\text{sr}} = 21$ kN/m ³

Base soil properties

Soil type	Very loose gravel
Soil density	$\gamma_{\text{b}} = 16$ kN/m ³
Presumed bearing capacity	$P_{\text{bearing}} = 75$ kN/m ²

Loading details

Variable surcharge load	Surcharge _Q = 10 kN/m ²
Vertical line load at 1350 mm	$P_{\text{G1}} = 4.2$ kN/m
	$P_{\text{Q1}} = 7$ kN/m



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28 HARROW ROAD, NEWPORT

Project
MAES Y COED GUEST HOUSE, MERTHYR TYDFIL, CF47

Section
Retaining Wall

Calc. by
ZR

Date
05-12-2024

Chk'd by
ZR

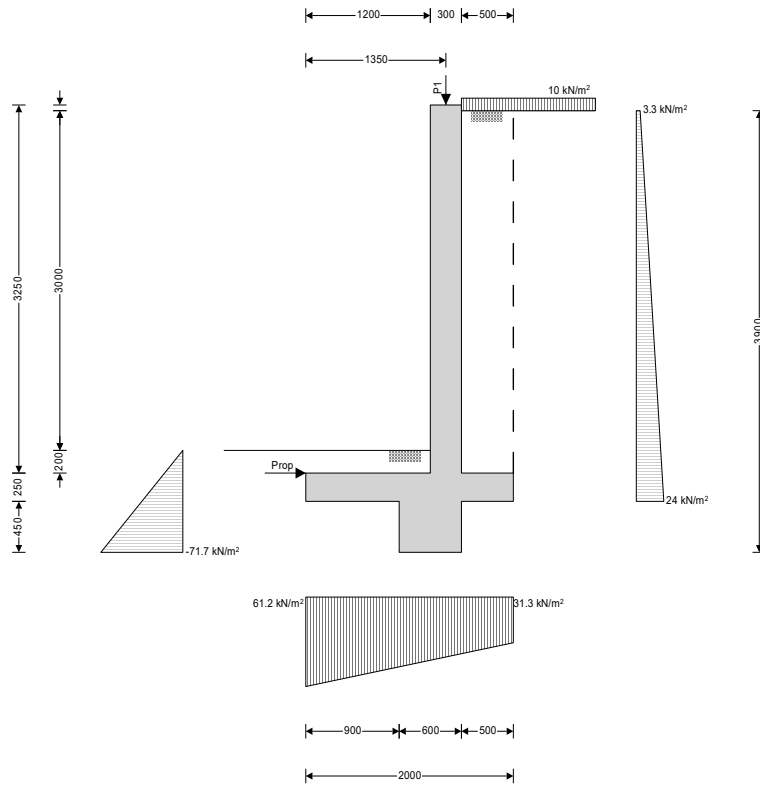
Date
05-12-2024

Job Ref.

Sheet no./rev.
01/ 2

App'd by
ZR

Date
05-12-2024



General arrangement - sketch pressures relate to bearing check

Calculate retaining wall geometry

Base length

$$l_{\text{base}} = l_{\text{toe}} + t_{\text{stem}} + l_{\text{heel}} = 2000 \text{ mm}$$

Base height

$$h_{\text{base}} = t_{\text{base}} + d_{\text{key}} = 700 \text{ mm}$$

Moist soil height

$$h_{\text{moist}} = h_{\text{soil}} = 3200 \text{ mm}$$

Length of surcharge load

$$l_{\text{sur}} = l_{\text{heel}} = 500 \text{ mm}$$

- Distance to vertical component

$$x_{\text{sur}_v} = l_{\text{base}} - l_{\text{heel}} / 2 = 1750 \text{ mm}$$

Effective height of wall

$$h_{\text{eff}} = h_{\text{base}} + d_{\text{cover}} + h_{\text{ret}} = 3900 \text{ mm}$$

- Distance to horizontal component

$$x_{\text{sur}_h} = h_{\text{eff}} / 2 - d_{\text{key}} = 1500 \text{ mm}$$

- Distance to horizontal component above key

$$x_{\text{sur}_h_a} = (h_{\text{eff}} - d_{\text{key}}) / 2 = 1725 \text{ mm}$$

Area of wall stem

$$A_{\text{stem}} = h_{\text{stem}} \times t_{\text{stem}} = 0.975 \text{ m}^2$$

- Distance to vertical component

$$x_{\text{stem}} = l_{\text{toe}} + t_{\text{stem}} / 2 = 1350 \text{ mm}$$

Area of wall base

$$A_{\text{base}} = l_{\text{base}} \times t_{\text{base}} + d_{\text{key}} \times t_{\text{key}} = 0.77 \text{ m}^2$$

- Distance to vertical component

$$x_{\text{base}} = (l_{\text{base}}^2 \times t_{\text{base}} / 2 + d_{\text{key}} \times t_{\text{key}} \times (p_{\text{key}} + t_{\text{key}} / 2)) / A_{\text{base}} = 1070 \text{ mm}$$

Area of moist soil

$$A_{\text{moist}} = h_{\text{moist}} \times l_{\text{heel}} = 1.6 \text{ m}^2$$

- Distance to vertical component

$$x_{\text{moist}_v} = l_{\text{base}} - (h_{\text{moist}} \times l_{\text{heel}}^2 / 2) / A_{\text{moist}} = 1750 \text{ mm}$$

- Distance to horizontal component

$$x_{\text{moist}_h} = h_{\text{eff}} / 3 - d_{\text{key}} = 850 \text{ mm}$$

- Distance to horizontal component above key

$$x_{\text{moist}_h_a} = (h_{\text{eff}} - d_{\text{key}}) / 3 = 1150 \text{ mm}$$

Area of base soil

$$A_{\text{pass}} = d_{\text{cover}} \times l_{\text{toe}} = 0.24 \text{ m}^2$$

- Distance to vertical component

$$x_{\text{pass}_v} = l_{\text{base}} - (d_{\text{cover}} \times l_{\text{toe}} \times (l_{\text{base}} - l_{\text{toe}} / 2)) / A_{\text{pass}} = 600 \text{ mm}$$

- Distance to horizontal component

$$x_{\text{pass}_h} = (d_{\text{cover}} + h_{\text{base}}) / 3 - d_{\text{key}} = -150 \text{ mm}$$



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Area of excavated base soil

$$A_{exc} = h_{pass} \times l_{toe} = \mathbf{0.24 \text{ m}^2}$$

- Distance to vertical component

$$x_{exc_v} = l_{base} - (h_{pass} \times l_{toe} \times (l_{base} - l_{toe} / 2)) / A_{exc} = \mathbf{600 \text{ mm}}$$

- Distance to horizontal component

$$x_{exc_h} = (h_{pass} + h_{base}) / 3 - d_{key} = \mathbf{-150 \text{ mm}}$$

Retained soil properties

Design moist density

$$\gamma_{mr}' = \gamma_{mr} / \gamma_f = \mathbf{18 \text{ kN/m}^3}$$

Design saturated density

$$\gamma_{sr}' = \gamma_{sr} / \gamma_f = \mathbf{21 \text{ kN/m}^3}$$

Base soil properties

Design soil density

$$\gamma_b' = \gamma_b / \gamma_f = \mathbf{16 \text{ kN/m}^3}$$

Soil coefficients

Coefficient of friction to back of wall

$$K_{fr} = \mathbf{0.325}$$

Coefficient of friction to front of wall

$$K_{fb} = \mathbf{0.325}$$

Coefficient of friction beneath base

$$K_{fbb} = \mathbf{0.325}$$

Active pressure coefficient

$$K_A = \mathbf{0.333}$$

Passive pressure coefficient

$$K_P = \mathbf{4.977}$$

Bearing pressure check

Vertical forces on wall

Wall stem

$$F_{stem} = A_{stem} \times \gamma_{stem} = \mathbf{24.4 \text{ kN/m}}$$

Wall base

$$F_{base} = A_{base} \times \gamma_{base} = \mathbf{19.3 \text{ kN/m}}$$

Surcharge load

$$F_{sur_v} = \text{Surcharge}_Q \times l_{heel} = \mathbf{5 \text{ kN/m}}$$

Line loads

$$F_{P_v} = P_{G1} + P_{Q1} = \mathbf{11.2 \text{ kN/m}}$$

Moist retained soil

$$F_{moist_v} = A_{moist} \times \gamma_{mr} = \mathbf{28.8 \text{ kN/m}}$$

Base soil

$$F_{pass_v} = A_{pass} \times \gamma_b = \mathbf{3.8 \text{ kN/m}}$$

Total

$$F_{total_v} = F_{stem} + F_{base} + F_{sur_v} + F_{P_v} + F_{moist_v} + F_{pass_v} = \mathbf{92.5 \text{ kN/m}}$$

Horizontal forces on wall

Surcharge load

$$F_{sur_h} = K_A \times \text{Surcharge}_Q \times (h_{eff} - d_{key}) = \mathbf{11.5 \text{ kN/m}}$$

Moist retained soil

$$F_{moist_h} = K_A \times \gamma_{mr} \times (h_{eff} - d_{key})^2 / 2 = \mathbf{35.7 \text{ kN/m}}$$

Base soil

$$F_{pass_h} = -K_P \times \gamma_b \times (d_{cover} + h_{base})^2 / 2 = \mathbf{-32.2 \text{ kN/m}}$$

Total

$$F_{total_h} = F_{sur_h} + F_{moist_h} + F_{pass_h} = \mathbf{15 \text{ kN/m}}$$

Moments on wall

Wall stem

$$M_{stem} = F_{stem} \times x_{stem} = \mathbf{32.9 \text{ kNm/m}}$$

Wall base

$$M_{base} = F_{base} \times x_{base} = \mathbf{20.6 \text{ kNm/m}}$$

Surcharge load

$$M_{sur} = F_{sur_v} \times x_{sur_v} - F_{sur_h} \times x_{sur_h} = \mathbf{-8.5 \text{ kNm/m}}$$

Line loads

$$M_P = (P_{G1} + P_{Q1}) \times p_1 = \mathbf{15.1 \text{ kNm/m}}$$

Moist retained soil

$$M_{moist} = F_{moist_v} \times x_{moist_v} - F_{moist_h} \times x_{moist_h} = \mathbf{20 \text{ kNm/m}}$$

Base soil

$$M_{pass} = F_{pass_v} \times x_{pass_v} = \mathbf{2.3 \text{ kNm/m}}$$

Total

$$M_{total} = M_{stem} + M_{base} + M_{sur} + M_P + M_{moist} + M_{pass} = \mathbf{82.5 \text{ kNm/m}}$$

Check bearing pressure

Propping force

$$F_{prop_base} = F_{total_h} = \mathbf{15 \text{ kN/m}}$$

Distance to reaction

$$\bar{x} = M_{total} / F_{total_v} = \mathbf{892 \text{ mm}}$$

Eccentricity of reaction

$$e = \bar{x} - l_{base} / 2 = \mathbf{-108 \text{ mm}}$$

Loaded length of base

$$l_{load} = l_{base} = \mathbf{2000 \text{ mm}}$$



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Bearing pressure at toe

$$q_{toe} = F_{total_v} / l_{base} \times (1 - 6 \times e / l_{base}) = \mathbf{61.2 \text{ kN/m}^2}$$

Bearing pressure at heel

$$q_{heel} = F_{total_v} / l_{base} \times (1 + 6 \times e / l_{base}) = \mathbf{31.3 \text{ kN/m}^2}$$

Factor of safety

$$FoS_{bp} = P_{bearing} / \max(q_{toe}, q_{heel}) = \mathbf{1.225}$$

PASS - Allowable bearing pressure exceeds maximum applied bearing pressure

RETAINING WALL DESIGN

In accordance with EN1992-1-1:2004 incorporating Corrigendum dated January 2008 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 2.9.22

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

Concrete strength class	C32/40
Characteristic compressive cylinder strength	$f_{ck} = \mathbf{32 \text{ N/mm}^2}$
Characteristic compressive cube strength	$f_{ck,cube} = \mathbf{40 \text{ N/mm}^2}$
Mean value of compressive cylinder strength	$f_{cm} = f_{ck} + 8 \text{ N/mm}^2 = \mathbf{40 \text{ N/mm}^2}$
Mean value of axial tensile strength	$f_{ctm} = 0.3 \text{ N/mm}^2 \times (f_{ck} / 1 \text{ N/mm}^2)^{2/3} = \mathbf{3.0 \text{ N/mm}^2}$
5% fractile of axial tensile strength	$f_{ctk,0.05} = 0.7 \times f_{ctm} = \mathbf{2.1 \text{ N/mm}^2}$
Secant modulus of elasticity of concrete	$E_{cm} = 22 \text{ kN/mm}^2 \times (f_{cm} / 10 \text{ N/mm}^2)^{0.3} = \mathbf{33346 \text{ N/mm}^2}$
Partial factor for concrete - Table 2.1N	$\gamma_C = \mathbf{1.50}$
Compressive strength coefficient - cl.3.1.6(1)	$\alpha_{cc} = \mathbf{0.85}$
Design compressive concrete strength - exp.3.15	$f_{cd} = \alpha_{cc} \times f_{ck} / \gamma_C = \mathbf{18.1 \text{ N/mm}^2}$
Maximum aggregate size	$h_{agg} = \mathbf{20 \text{ mm}}$
Ultimate strain - Table 3.1	$\epsilon_{cu2} = \mathbf{0.0035}$
Shortening strain - Table 3.1	$\epsilon_{cu3} = \mathbf{0.0035}$
Effective compression zone height factor	$\lambda = \mathbf{0.80}$
Effective strength factor	$\eta = \mathbf{1.00}$
Bending coefficient k_1	$K_1 = \mathbf{0.40}$
Bending coefficient k_2	$K_2 = 1.00 \times (0.6 + 0.0014/\epsilon_{cu2}) = \mathbf{1.00}$
Bending coefficient k_3	$K_3 = \mathbf{0.40}$
Bending coefficient k_4	$K_4 = 1.00 \times (0.6 + 0.0014/\epsilon_{cu2}) = \mathbf{1.00}$

Reinforcement details

Characteristic yield strength of reinforcement	$f_{yk} = \mathbf{500 \text{ N/mm}^2}$
Modulus of elasticity of reinforcement	$E_s = \mathbf{200000 \text{ N/mm}^2}$
Partial factor for reinforcing steel - Table 2.1N	$\gamma_S = \mathbf{1.15}$
Design yield strength of reinforcement	$f_{yd} = f_{yk} / \gamma_S = \mathbf{435 \text{ N/mm}^2}$

Cover to reinforcement

Front face of stem	$c_{sf} = \mathbf{40 \text{ mm}}$
Rear face of stem	$c_{sr} = \mathbf{50 \text{ mm}}$
Top face of base	$c_{bt} = \mathbf{50 \text{ mm}}$
Bottom face of base	$c_{bb} = \mathbf{75 \text{ mm}}$



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Tension reinforcement provided

16 dia.bars @ 200 c/c

Area of tension reinforcement provided

$A_{sr,prov} = \pi \times \phi_{sr}^2 / (4 \times s_{sr}) = 1005 \text{ mm}^2/\text{m}$

Minimum area of reinforcement - exp.9.1N

$A_{sr,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 381 \text{ mm}^2/\text{m}$

Maximum area of reinforcement - cl.9.2.1.1(3)

$A_{sr,max} = 0.04 \times h = 12000 \text{ mm}^2/\text{m}$

$\max(A_{sr,req}, A_{sr,min}) / A_{sr,prov} = 0.695$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Deflection control - Section 7.4

Reference reinforcement ratio

$\rho_0 = \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} / 1000 = 0.006$

Required tension reinforcement ratio

$\rho = A_{sr,req} / d = 0.003$

Required compression reinforcement ratio

$\rho' = A_{sr,2,req} / d_2 = 0.000$

Structural system factor - Table 7.4N

$K_b = 0.4$

Reinforcement factor - exp.7.17

$K_s = \min(500 \text{ N/mm}^2 / (f_{yk} \times A_{sr,req} / A_{sr,prov}), 1.5) = 1.439$

Limiting span to depth ratio - exp.7.16.a

$\min(K_s \times K_b \times [11 + 1.5 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times \rho_0 / \rho + 3.2 \times \sqrt{(f_{ck} / 1 \text{ N/mm}^2)} \times (\rho_0 / \rho - 1)^{3/2}], 40 \times K_b) = 16$

Actual span to depth ratio

$h_{stem} / d = 13.4$

PASS - Span to depth ratio is less than deflection control limit

Crack control - Section 7.3

Limiting crack width

$w_{max} = 0.3 \text{ mm}$

Variable load factor - EN1990 – Table A1.1

$\psi/2 = 0.6$

Serviceability bending moment

$M_{sls} = 43 \text{ kNm/m}$

Tensile stress in reinforcement

$\sigma_s = M_{sls} / (A_{sr,prov} \times z) = 186.1 \text{ N/mm}^2$

Load duration

Long term

Load duration factor

$k_t = 0.4$

Effective area of concrete in tension

$A_{c,eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$A_{c,eff} = 89917 \text{ mm}^2/\text{m}$

Mean value of concrete tensile strength

$f_{ct,eff} = f_{ctm} = 3.0 \text{ N/mm}^2$

Reinforcement ratio

$\rho_{p,eff} = A_{sr,prov} / A_{c,eff} = 0.011$

Modular ratio

$\alpha_e = E_s / E_{cm} = 5.998$

Bond property coefficient

$k_1 = 0.8$

Strain distribution coefficient

$k_2 = 0.5$

$k_3 = 3.4$

$k_4 = 0.425$

Maximum crack spacing - exp.7.11

$s_{r,max} = k_3 \times c_{sr} + k_1 \times k_2 \times k_4 \times \phi_{sr} / \rho_{p,eff} = 413 \text{ mm}$

Maximum crack width - exp.7.8

$w_k = s_{r,max} \times \max(\sigma_s - k_t \times (f_{ct,eff} / \rho_{p,eff}) \times (1 + \alpha_e \times \rho_{p,eff}), 0.6 \times \sigma_s) / E_s$

$w_k = 0.231 \text{ mm}$

$w_k / w_{max} = 0.769$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force

$V = 57.5 \text{ kN/m}$

$C_{Rd,c} = 0.18 / \gamma_c = 0.120$

$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 1.909$

Longitudinal reinforcement ratio

$\rho_l = \min(A_{sr,prov} / d, 0.02) = 0.004$



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Design shear resistance - exp.6.2a & 6.2b

$$V_{min} = 0.035 N^{1/2}/mm \times k^{3/2} \times f_{ck}^{0.5} = \mathbf{0.522 N/mm^2}$$

$$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 N^2/mm^4 \times \rho_l \times f_{ck})^{1/3}, V_{min}) \times d$$

$$V_{Rd,c} = \mathbf{131.3 kN/m}$$

$$V / V_{Rd,c} = \mathbf{0.438}$$

PASS - Design shear resistance exceeds design shear force

Horizontal reinforcement parallel to face of stem - Section 9.6

Minimum area of reinforcement – cl.9.6.3(1) $A_{sx,req} = \max(0.25 \times A_{sr,prov}, 0.001 \times t_{stem}) = \mathbf{300 mm^2/m}$

Maximum spacing of reinforcement – cl.9.6.3(2) $S_{sx,max} = \mathbf{400 mm}$

Transverse reinforcement provided 10 dia.bars @ 200 c/c

Area of transverse reinforcement provided $A_{sx,prov} = \pi \times \phi_{sx}^2 / (4 \times S_{sx}) = \mathbf{393 mm^2/m}$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Check base design at toe

Depth of section $h = \mathbf{250 mm}$

Rectangular section in flexure - Section 6.1

Design bending moment combination 1 $M = \mathbf{50.7 kNm/m}$

Depth to tension reinforcement $d = h - C_{bb} - \phi_{bb} / 2 = \mathbf{167 mm}$

$$K = M / (d^2 \times f_{ck}) = \mathbf{0.057}$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - K_1) / (2 \times K_2)) \times (\lambda \times (\delta - K_1) / (2 \times K_2))$$

$$K' = \mathbf{0.207}$$

K' > K - No compression reinforcement is required

Lever arm $z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}, 0.95) \times d = \mathbf{158 mm}$

Depth of neutral axis $x = 2.5 \times (d - z) = \mathbf{22 mm}$

Area of tension reinforcement required $A_{bb,req} = M / (f_{yd} \times z) = \mathbf{737 mm^2/m}$

Tension reinforcement provided 16 dia.bars @ 200 c/c

Area of tension reinforcement provided $A_{bb,prov} = \pi \times \phi_{bb}^2 / (4 \times S_{bb}) = \mathbf{1005 mm^2/m}$

Minimum area of reinforcement - exp.9.1N $A_{bb,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = \mathbf{263 mm^2/m}$

Maximum area of reinforcement - cl.9.2.1.1(3) $A_{bb,max} = 0.04 \times h = \mathbf{10000 mm^2/m}$

$$\max(A_{bb,req}, A_{bb,min}) / A_{bb,prov} = \mathbf{0.733}$$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width $w_{max} = \mathbf{0.3 mm}$

Variable load factor - EN1990 – Table A1.1 $\psi_2 = \mathbf{0.6}$

Serviceability bending moment $M_{sls} = \mathbf{31.2 kNm/m}$

Tensile stress in reinforcement $\sigma_s = M_{sls} / (A_{bb,prov} \times z) = \mathbf{196.5 N/mm^2}$

Load duration Long term

Load duration factor $k_t = \mathbf{0.4}$

Effective area of concrete in tension $A_{c,eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$

$$A_{c,eff} = \mathbf{75969 mm^2/m}$$

Mean value of concrete tensile strength $f_{ct,eff} = f_{ctm} = \mathbf{3.0 N/mm^2}$

Reinforcement ratio $\rho_{p,eff} = A_{bb,prov} / A_{c,eff} = \mathbf{0.013}$

Modular ratio $\alpha_e = E_s / E_{cm} = \mathbf{5.998}$

Bond property coefficient $k_1 = \mathbf{0.8}$



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Strain distribution coefficient

$$k_2 = 0.5$$

$$k_3 = 3.4$$

$$k_4 = 0.425$$

Maximum crack spacing - exp.7.11

$$s_{r,max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_{bb} / \rho_{p,eff} = 461 \text{ mm}$$

Maximum crack width - exp.7.8

$$w_k = s_{r,max} \times \max(\sigma_s - k_t \times (f_{ct,eff} / \rho_{p,eff}) \times (1 + \alpha_e \times \rho_{p,eff}), 0.6 \times \sigma_s) / E_s$$

$$w_k = 0.271 \text{ mm}$$

$$w_k / w_{max} = 0.905$$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force

$$V = 80.2 \text{ kN/m}$$

$$C_{Rd,c} = 0.18 / \gamma_c = 0.120$$

$$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 2.000$$

Longitudinal reinforcement ratio

$$\rho_l = \min(A_{bb,prov} / d, 0.02) = 0.006$$

$$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.560 \text{ N/mm}^2$$

Design shear resistance - exp.6.2a & 6.2b

$$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$$

$$V_{Rd,c} = 107.4 \text{ kN/m}$$

$$V / V_{Rd,c} = 0.746$$

PASS - Design shear resistance exceeds design shear force

Check base design at heel

Depth of section

$$h = 250 \text{ mm}$$

Rectangular section in flexure - Section 6.1

Design bending moment combination 1

$$M = 5.9 \text{ kNm/m}$$

Depth to tension reinforcement

$$d = h - c_{bt} - \phi_{bt} / 2 = 195 \text{ mm}$$

$$K = M / (d^2 \times f_{ck}) = 0.005$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - K_1) / (2 \times K_2)) \times (\lambda \times (\delta - K_1) / (2 \times K_2))$$

$$K' = 0.207$$

K' > K - No compression reinforcement is required

Lever arm

$$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}, 0.95) \times d = 185 \text{ mm}$$

Depth of neutral axis

$$x = 2.5 \times (d - z) = 24 \text{ mm}$$

Area of tension reinforcement required

$$A_{bt,req} = M / (f_{yd} \times z) = 73 \text{ mm}^2/\text{m}$$

Tension reinforcement provided

$$10 \text{ dia.bars @ } 200 \text{ c/c}$$

Area of tension reinforcement provided

$$A_{bt,prov} = \pi \times \phi_{bt}^2 / (4 \times s_{bt}) = 393 \text{ mm}^2/\text{m}$$

Minimum area of reinforcement - exp.9.1N

$$A_{bt,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 307 \text{ mm}^2/\text{m}$$

Maximum area of reinforcement - cl.9.2.1.1(3)

$$A_{bt,max} = 0.04 \times h = 10000 \text{ mm}^2/\text{m}$$

$$\max(A_{bt,req}, A_{bt,min}) / A_{bt,prov} = 0.781$$

PASS - Area of reinforcement provided is greater than area of reinforcement required

Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width

$$w_{max} = 0.3 \text{ mm}$$

Variable load factor - EN1990 – Table A1.1

$$\psi/2 = 0.6$$

Serviceability bending moment

$$M_{sls} = 3.6 \text{ kNm/m}$$

Tensile stress in reinforcement

$$\sigma_s = M_{sls} / (A_{bt,prov} \times z) = 49.8 \text{ N/mm}^2$$

Load duration

Long term



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Load duration factor

$$k_t = 0.4$$

Effective area of concrete in tension

$$A_{c,eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$$

$$A_{c,eff} = 75208 \text{ mm}^2/\text{m}$$

Mean value of concrete tensile strength

$$f_{ct,eff} = f_{ctm} = 3.0 \text{ N/mm}^2$$

Reinforcement ratio

$$\rho_{p,eff} = A_{bt,prov} / A_{c,eff} = 0.005$$

Modular ratio

$$\alpha_e = E_s / E_{cm} = 5.998$$

Bond property coefficient

$$k_1 = 0.8$$

Strain distribution coefficient

$$k_2 = 0.5$$

$$k_3 = 3.4$$

$$k_4 = 0.425$$

Maximum crack spacing - exp.7.11

$$s_{r,max} = k_3 \times c_{bt} + k_1 \times k_2 \times k_4 \times \phi_{bt} / \rho_{p,eff} = 496 \text{ mm}$$

Maximum crack width - exp.7.8

$$w_k = s_{r,max} \times \max(\sigma_s - k_t \times (f_{ct,eff} / \rho_{p,eff}) \times (1 + \alpha_e \times \rho_{p,eff}), 0.6 \times \sigma_s) / E_s$$

$$w_k = 0.074 \text{ mm}$$

$$w_k / w_{max} = 0.247$$

PASS - Maximum crack width is less than limiting crack width

Rectangular section in shear - Section 6.2

Design shear force

$$V = 31.8 \text{ kN/m}$$

$$C_{Rd,c} = 0.18 / \gamma_c = 0.120$$

$$k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 2.000$$

Longitudinal reinforcement ratio

$$\rho_l = \min(A_{bt,prov} / d, 0.02) = 0.002$$

$$v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.560 \text{ N/mm}^2$$

Design shear resistance - exp.6.2a & 6.2b

$$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$$

$$V_{Rd,c} = 109.2 \text{ kN/m}$$

$$V / V_{Rd,c} = 0.291$$

PASS - Design shear resistance exceeds design shear force

Check key design

Depth of section

$$h = 600 \text{ mm}$$

Rectangular section in flexure - Section 6.1

Design bending moment combination 1

$$M = 3.1 \text{ kNm/m}$$

Depth to tension reinforcement

$$d = h - c_{bb} - \phi_k / 2 = 517 \text{ mm}$$

$$K = M / (d^2 \times f_{ck}) = 0.000$$

$$K' = (2 \times \eta \times \alpha_{cc} / \gamma_c) \times (1 - \lambda \times (\delta - K_1) / (2 \times K_2)) \times (\lambda \times (\delta - K_1) / (2 \times K_2))$$

$$K' = 0.207$$

K' > K - No compression reinforcement is required

Lever arm

$$z = \min(0.5 + 0.5 \times (1 - 2 \times K / (\eta \times \alpha_{cc} / \gamma_c))^{0.5}, 0.95) \times d = 491 \text{ mm}$$

Depth of neutral axis

$$x = 2.5 \times (d - z) = 65 \text{ mm}$$

Area of tension reinforcement required

$$A_{k,req} = M / (f_{yd} \times z) = 15 \text{ mm}^2/\text{m}$$

Tension reinforcement provided

$$16 \text{ dia.bars @ } 200 \text{ c/c}$$

Area of tension reinforcement provided

$$A_{k,prov} = \pi \times \phi_k^2 / (4 \times s_k) = 1005 \text{ mm}^2/\text{m}$$

Minimum area of reinforcement - exp.9.1N

$$A_{k,min} = \max(0.26 \times f_{ctm} / f_{yk}, 0.0013) \times d = 813 \text{ mm}^2/\text{m}$$

Maximum area of reinforcement - cl.9.2.1.1(3)

$$A_{k,max} = 0.04 \times h = 24000 \text{ mm}^2/\text{m}$$

$$\max(A_{k,req}, A_{k,min}) / A_{k,prov} = 0.809$$

PASS - Area of reinforcement provided is greater than area of reinforcement required



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Library item: Rectangular single output

Crack control - Section 7.3

Limiting crack width	$w_{max} = 0.3 \text{ mm}$
Variable load factor - EN1990 – Table A1.1	$\psi/2 = 0.6$
Serviceability bending moment	$M_{sls} = 2.1 \text{ kNm/m}$
Tensile stress in reinforcement	$\sigma_s = M_{sls} / (A_{k,prov} \times z) = 4.3 \text{ N/mm}^2$
Load duration	Long term
Load duration factor	$k_t = 0.4$
Effective area of concrete in tension	$A_{c,eff} = \min(2.5 \times (h - d), (h - x) / 3, h / 2)$ $A_{c,eff} = 178458 \text{ mm}^2/\text{m}$
Mean value of concrete tensile strength	$f_{ct,eff} = f_{ctm} = 3.0 \text{ N/mm}^2$
Reinforcement ratio	$\rho_{p,eff} = A_{k,prov} / A_{c,eff} = 0.006$
Modular ratio	$\alpha_e = E_s / E_{cm} = 5.998$
Bond property coefficient	$k_1 = 0.8$
Strain distribution coefficient	$k_2 = 0.5$ $k_3 = 3.4$ $k_4 = 0.425$
Maximum crack spacing - exp.7.11	$s_{r,max} = k_3 \times c_{bb} + k_1 \times k_2 \times k_4 \times \phi_k / \rho_{p,eff} = 738 \text{ mm}$
Maximum crack width - exp.7.8	$w_k = s_{r,max} \times \max(\sigma_s - k_t \times (f_{ct,eff} / \rho_{p,eff}) \times (1 + \alpha_e \times \rho_{p,eff}), 0.6 \times \sigma_s) / E_s$ $w_k = 0.01 \text{ mm}$ $w_k / w_{max} = 0.032$
PASS - Maximum crack width is less than limiting crack width	

Rectangular section in shear - Section 6.2

Design shear force	$V = 13.9 \text{ kN/m}$ $C_{Rd,c} = 0.18 / \gamma_c = 0.120$ $k = \min(1 + \sqrt{(200 \text{ mm} / d)}, 2) = 1.622$
Longitudinal reinforcement ratio	$\rho_l = \min(A_{k,prov} / d, 0.02) = 0.002$ $v_{min} = 0.035 \text{ N}^{1/2}/\text{mm} \times k^{3/2} \times f_{ck}^{0.5} = 0.409 \text{ N/mm}^2$
Design shear resistance - exp.6.2a & 6.2b	$V_{Rd,c} = \max(C_{Rd,c} \times k \times (100 \text{ N}^2/\text{mm}^4 \times \rho_l \times f_{ck})^{1/3}, v_{min}) \times d$ $V_{Rd,c} = 211.4 \text{ kN/m}$ $V / V_{Rd,c} = 0.066$
PASS - Design shear resistance exceeds design shear force	

Secondary transverse reinforcement to base - Section 9.3

Minimum area of reinforcement – cl.9.3.1.1(2)	$A_{bx,req} = 0.2 \times A_{bb,prov} = 201 \text{ mm}^2/\text{m}$
Maximum spacing of reinforcement – cl.9.3.1.1(3)	$s_{bx,max} = 450 \text{ mm}$
Transverse reinforcement provided	10 dia.bars @ 200 c/c
Area of transverse reinforcement provided	$A_{bx,prov} = \pi \times \phi_{bx}^2 / (4 \times s_{bx}) = 393 \text{ mm}^2/\text{m}$
PASS - Area of reinforcement provided is greater than area of reinforcement required	



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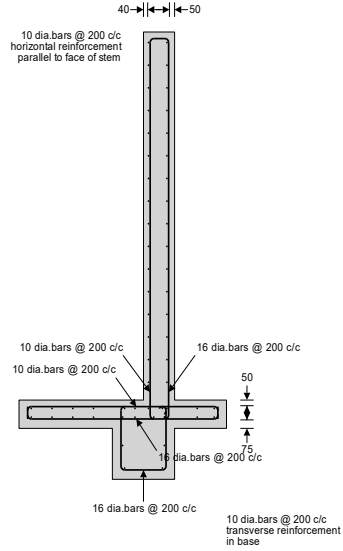
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Reinforcement details