



STRUCTURAL CALCULATIONS

AUX843

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Project name: **Proposed Residential Dwelling**

Client name: **Philip Harris (Arch) TBC**

Property address: **1a Blaendowlais, Dowlais,
Merthyr Tydfil,
CF48 3RB**

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Address: **50 Quarry Row,
Merthyr Tydfil,
CF47 8PS**

Date of preparation: **24/08/2026**

Job reference number: **AUX843**



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1 Introduction

In accordance with instructions received from the client I have carried out a Structural calculations for proposed residential dwelling.

2 Location of the Property

The property is situated at

1a Blaendowlais, Dowlais,
Merthyr Tydfil,
CF48 3RB

See below the satellite view from Google Maps.





3 Construction Method Statement: Timber Cut Roof Structure

Before starting any of the below works please contact Building Control for guidance regarding Building Control Approval

Reference Standard: BS EN 1995-1-1 (Eurocode 5) & BS 8103-3

1.0 General Specification

- **Timber Grade:** All structural timber (rafters, purlins, ceiling joists, wall plates) to be Strength Class **C24** (unless specified otherwise in calculations) to BS EN 338.
- **Preservative:** All timbers to be pressure treated with preservative suitable for Use Class 2 (internal risk of wetting). Cut ends must be re-treated on site.
- **Moisture Content:** Moisture content at time of erection not to exceed 20%.

2.0 Construction Sequence (Step-by-Step)

Step 1: Wall Plate Installation

- Bed timber wall plates (typically 100x 75mm) onto a full bed of mortar (1:3 cement:sand) along the top of the inner leaf of load-bearing masonry.²
- Ensure plates are level and aligned.
- Join lengths of wall plate using **half-lap joints** (min. 100mm lap), fully nailed with 2no. Ø4.0 x 90mm nails.
- **Restraint:** Install vertical holding down straps at this stage (see Section 3.0 for detail).

Step 2: Ceiling Joists (Ties)

- Install ceiling joists spanning between load-bearing walls.
- Fix joists to the wall plate using skew-nailing or truss clips.
- **Central Binder:** Where joist spans require, install a continuous runner (binder) centrally along the top of the joists to maintain spacing and prevent buckling.

Step 3: Primary Support Structure (Purlins & Ridge)

- Install Purlins as per structural calculations. Purlins must be supported by load-bearing walls or designed struts.
- **Strutting:** If struts are used to support purlins, they must bear directly onto load-bearing walls below (not mid-span of joists) and be effectively cleated to the purlin.
- Install the Ridge Board. This provides the apex fixing point for rafters.

Step 4: Rafter Installation

- Cut rafters with a "birds-mouth" notch to seat securely onto the wall plate. The depth of the notch must not exceed 1/3 of the rafter depth.
- Fix rafters at the top (ridge) and bottom (wall plate):
 - **Ridge:** Skew nail to ridge board.



- **Wall Plate:** Skew nail to wall plate.
- **Opposing Pairs:** Rafters must be installed in opposing pairs to prevent lateral thrust on the ridge board before the roof is fully triangulated.

Step 5: Final Restraint & Bracing

- Install lateral restraint straps to gable walls (see Section 3.0).
- Install diagonal and longitudinal bracing (see Section 4.0).

3.0 Metal Strapping Specifications (Restraint)

This section details the mechanical fixing required to resist wind uplift and provide lateral stability to masonry walls.

3.1 Vertical Holding Down Straps (Wind Uplift)

- **Purpose:** To prevent the roof lifting off the wall plate under wind loading.
- **Specification:** Galvanised steel straps, minimum cross-section **30 x 2.5mm**.
- **Spacing:** Maximum **2.0m centres** along the wall plate.
- **Fixing:**
 - Top: Fixed to the wall plate with min. 3no. 30mm x 3.75mm square twist nails.
 - Bottom: Fixed to the masonry vertical face. The strap must extend at least **1.0m** down the masonry. Fix using min. 4no. stainless steel screws (plugged) or hardened masonry nails.
 - *Note: The lowest fixing must be within 150mm of the bottom of the strap.*

3.2 Horizontal Lateral Restraint Straps (Gable Stability)

- **Purpose:** To prevent gable walls bowing out by tying them into the roof structure.
- **Specification:** Heavy-duty galvanised steel straps, minimum cross-section **30 x 5.0mm**.
- **Spacing:** Maximum **2.0m centres** along the rake of the gable.
- **Length:** Strap must be long enough to span across at least **3no. rafters**.
- **Installation:**
 - The "cranked" end of the strap is to be securely embedded in the mortar joint or screwed directly to the masonry face (if using a straight strap with a separate anchor).
 - The strap arm must be fixed to **each** of the 3 rafters it crosses.
 - **Packing/Noggings:** Solid timber noggings must be installed between the rafters at the strap position to prevent the rafters twisting. The strap fixes into these noggings or solid blocks.



4.0 Bracing Requirements

Unless rigid sarking (e.g., OSB or Plywood) is applied over the entire roof area, the following timber bracing is required to prevent racking.

4.1 Longitudinal Bracing

- Provide a continuous timber binder (min 100 x25mm) running the full length of the roof at node points (e.g., at the level of the purlin or ridge).
- This binds all rafters together ensuring they act as a single unit.

4.2 Diagonal Rafter Bracing

- **Location:** Underside of rafters, running from the eaves to the ridge at approximately 45°.
- **Extent:** At least one diagonal brace on each pitch of the roof. For wide roofs, provide criss-cross (X) bracing.
- **Fixing:** Nail to the underside of every rafter it crosses with 2no. 3.35 x 65mm nails.

4.3 Chevron Bracing (For spans > 8m or high wind zones)

- If specified in calculations, install V-shaped bracing across the rafter plane to effectively transfer wind loads to the supporting structure.



4 Sequence of Operations: Reinforced Blockwork and Solid Blockwork Retaining Walls

Project Component: Reinforced Blockwork and Solid Blockwork Retaining Walls

Design Basis: All works shall be carried out strictly in accordance with the Structural Engineer's drawings, calculations and approved construction details.

Building Control: Before commencement of the works, Building Control should be contacted as required to confirm inspection requirements and Building Control approval.

Project Notes

- All works shall be carried out in accordance with the Structural Engineer's drawings and calculations.
- The contractor shall verify all dimensions, levels and existing site conditions prior to commencement.
- The retaining walls shall be constructed to the locations, thicknesses, heights and foundation arrangements shown on the Structural Engineer's drawings.
- The contractor shall ensure that suitable temporary works and safe working arrangements are provided throughout the construction.
- No structural wall, reinforcement, foundation or connection shall be altered without prior approval from the Structural Engineer.
- The excavation and retaining wall construction sequence shall be arranged to maintain the stability of the surrounding ground and adjacent structures.

1. Site Preparation and Setting Out

Before commencing excavation, the retaining wall locations and levels shall be established.

- **Setting Out:** Set out the retaining wall positions in accordance with the approved structural and architectural drawings.
- **Existing Services:** Identify existing underground services within and adjacent to the proposed excavation area before excavation begins.
- **Levels:** Confirm existing ground levels, proposed finished levels and retaining wall heights.
- **Inspection:** Check the existing ground conditions and notify the Structural Engineer of any unexpected ground conditions encountered.
- **Access:** Provide safe access and suitable working areas around the proposed retaining wall construction.



2. Excavation and Formation

- Excavate to the required foundation dimensions and levels shown on the Structural Engineer's drawings.
- Excavation shall be carried out in a controlled manner to avoid undermining adjacent structures, walls or foundations.
- Keep the excavation free from standing water and loose or disturbed material.
- The formation level shall be inspected before placing concrete.
- Any soft, loose or unsuitable material encountered at formation level shall be reported to the Structural Engineer before proceeding.
- Provide suitable temporary support or battering to excavation faces where required for safe working.

3. Foundation Construction

The retaining wall foundations shall be constructed in accordance with the structural drawings.

- **Formation:** Trim and compact the formation to provide a firm and level base.
- **Blinding:** Provide concrete blinding where specified to provide a clean and level surface for reinforcement and foundation construction.
- **Reinforcement:** Install foundation reinforcement in accordance with the Structural Engineer's reinforcement details.
- **Cover:** Ensure the specified concrete cover to reinforcement is maintained using suitable spacers and chairs.
- **Formwork:** Install formwork where required to achieve the specified foundation dimensions.
- **Inspection:** The reinforcement and formwork shall be inspected prior to concrete placement.
- **Concrete:** Place the specified concrete carefully to avoid displacement of reinforcement.
- **Compaction:** Compact the concrete adequately to prevent voids and honeycombing.
- **Curing:** Protect and cure the concrete in accordance with the concrete specification.

4. Construction of RW1 – Reinforced Blockwork Retaining Wall

RW1 is identified on the structural drawing as **215 mm reinforced blockwork**, with the retaining wall shown at approximately **1400 mm retained height**.

- **Blockwork:** Construct the wall using the specified reinforced blockwork units.
- **Setting Out:** Set out the wall accurately on the completed foundation.
- **Mortar:** Use the specified mortar appropriate to the blockwork and structural design.



- **Reinforcement:** Install vertical and horizontal reinforcement in accordance with the Structural Engineer's drawings.
- **Alignment:** Ensure the blockwork remains plumb, level and correctly aligned throughout construction.
- **Joints:** Maintain consistent and properly filled mortar joints.
- **Grouting:** Where the structural design requires reinforced cores to be filled, grout the specified blockwork cores in accordance with the Structural Engineer's details.
- **Connections:** Ensure the wall reinforcement is adequately connected to the foundation reinforcement as detailed.
- **Curing:** Allow sufficient time for mortar/grout to achieve the required strength before applying significant lateral loading or commencing backfilling.

5. Construction of RW2 – Solid Blockwork Retaining Wall

RW2 is identified on the structural drawing as **330 mm solid blockwork**, with an approximately **990 mm retained height** shown.

- **Blockwork:** Construct the retaining wall using the specified solid blockwork units.
- **Setting Out:** Position the wall accurately on the completed foundation.
- **Mortar:** Use the specified structural mortar.
- **Bonding:** Construct the blockwork in the required bond and maintain proper alignment.
- **Alignment:** Check the wall regularly to ensure it remains plumb, level and correctly positioned.
- **Joints:** Ensure all joints are adequately filled and finished.
- **Connections:** Provide any required restraint, reinforcement or connection to the foundation in accordance with the Structural Engineer's drawings.
- **Curing:** Allow sufficient curing time before commencing backfilling.

6. Drainage and Waterproofing

Adequate drainage shall be provided behind the retaining walls where specified by the design.

- Install the specified drainage system behind the retaining wall.
- Provide suitable free-draining granular material where shown on the drawings or specification.
- Install drainage pipes, outlets and associated fittings where required.
- Ensure drainage outlets remain unobstructed.
- Provide waterproofing or protection to the rear face of the wall where specified.



- Prevent water from accumulating behind the retaining wall.

7. Backfilling

Backfilling shall only commence after the retaining wall and associated foundation have achieved the required strength.

- **Approval:** Confirm that the wall is sufficiently cured and stable before commencing backfilling.
- **Material:** Use suitable selected fill material as specified.
- **Placement:** Place backfill in controlled layers.
- **Compaction:** Compact each layer carefully using suitable equipment.
- **Wall Protection:** Avoid impact or excessive vibration against the retaining wall during compaction.
- **Drainage:** Maintain the designed drainage zone behind the wall throughout the backfilling operation.
- **Loading:** Do not place heavy plant or stored materials close to the retaining wall unless permitted by the Structural Engineer's design.

8. Completion and Inspection

Upon completion of the retaining wall construction, the following shall be checked:

1. Retaining wall location and dimensions are in accordance with the approved drawings.
2. Foundation dimensions and levels have been achieved.
3. Concrete foundation is adequately cured and free from significant defects.
4. RW1 reinforced blockwork is correctly constructed and reinforced as detailed.
5. RW2 solid blockwork is correctly constructed as detailed.
6. All specified reinforcement and structural connections are complete.
7. Wall alignment and levels have been checked.
8. Drainage and waterproofing provisions are complete where specified.
9. Backfill material and compaction comply with the specification.
10. No unexpected ground or structural conditions remain unresolved.
11. The completed retaining wall is ready for the associated groundworks and external works.



Contractor Disclaimer

The contractor is responsible for checking all dimensions, levels and site conditions prior to commencement of the works.

The contractor is responsible for providing suitable temporary works, excavation support, safe access and appropriate construction methods to maintain the stability of the retaining walls, surrounding ground and adjacent structures throughout the works.

Any discrepancy between the Structural Engineer's drawings and the actual site conditions shall be reported to the Structural Engineer before proceeding with the affected works.

No structural member, reinforcement, foundation or retaining wall detail shall be modified without prior approval from the Structural Engineer.

A handwritten signature in black ink, appearing to read 'Marcin Krawucki', with a long, sweeping horizontal stroke at the end.

Marcin Krawucki

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ICE Membership no.: 92007313



5 ***Appendices***

5.1. Calculations

5.2. Drawings

LOAD TAKEDOWN

B1

Pitched Roof			
DL	0.85kN/m2 x	2.70m =	2.30kN/m
LL	0.75kN/m2 x	2.70m =	2.03kN/m

TR1

Floor & staircase load			
DL	0.50kN/m2 x	1.70m =	0.85kN/m
LL	2.00kN/m2 x	1.70m =	3.40kN/m
Wall (Stud wall)			
DL	0.50kN/m2 x	2.10m =	1.05kN/m

TR2

TR1 point load @ 0.52m & 2.8m			
DL			1.77kN
LL			3.00kN

B2

Wall (Stud wall)			
DL	0.50kN/m2 x	1.20m =	0.60kN/m
Pitched Roof			
DL	0.85kN/m2 x	2.40m =	2.04kN/m
LL	0.75kN/m2 x	2.40m =	1.80kN/m
Floor			
DL	0.50kN/m2 x	2.00m =	1.00kN/m
LL	2.00kN/m2 x	2.00m =	4.00kN/m
Tie joists (Attic)			
DL	0.35kN/m2 x	0.60m =	0.21kN/m
LL	0.75kN/m2 x	0.60m =	0.45kN/m
Total dead UDL			= 3.85kN/m
Total Live UDL			= 6.25kN/m
TR2 point load @ 1.8m			
DL			2.21kN
LL			3.51kN

LOAD TAKEDOWN

TR3

Wall (Stud wall)			
DL	0.50kN/m2 x	2.35m =	1.18kN/m
Floor			
DL	0.50kN/m2 x	0.85m =	0.43kN/m
LL	2.00kN/m2 x	0.85m =	1.70kN/m
Total dead UDL			= 1.60kN/m
Total Live UDL			= 1.70kN/m

TR4

Staircase load			
DL	0.50kN/m2 x	0.70m =	0.35kN/m
LL	2.00kN/m2 x	0.70m =	1.40kN/m
TR3 point load @ 1.7m			
DL			1.50kN
LL			1.53kN

B3

Floor			
DL	0.50kN/m2 x	3.25m =	1.63kN/m
LL	2.00kN/m2 x	3.25m =	6.50kN/m
TR4 point load @ 1.8m			
DL			1.10kN
LL			2.20kN

L1

Wall (cavity wall)			
DL	2.00kN/m2 x	2.90m =	5.80kN/m
Floor			
DL	0.50kN/m2 x	2.00m =	1.00kN/m
LL	2.00kN/m2 x	2.00m =	4.00kN/m
Pitched Roof			
DL	0.85kN/m2 x	1.20m =	1.02kN/m
LL	0.75kN/m2 x	1.20m =	0.90kN/m

Tie joists (Attic)

DL	0.35kN/m ² x	0.60m =	0.21kN/m
LL	0.75kN/m ² x	0.60m =	0.45kN/m

Total dead UDL = **8.03kN/m**

Total Live UDL = **5.35kN/m**

Line 1**Wall (cavity wall)**

DL	4.00kN/m ² x	7.20m =	28.80kN/m
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B1 distributed load

DL	7.00kN /	7.10m =	0.99kN/m
LL	5.60kN /	7.10m =	0.79kN/m

B2 distributed load

DL	13.00kN /	7.10m =	1.83kN/m
LL	19.50kN /	7.10m =	2.75kN/m

B2 distributed load

DL	13.00kN /	7.10m =	1.83kN/m
LL	19.50kN /	7.10m =	2.75kN/m

B3 distributed load

DL	6.60kN /	5.20m =	1.27kN/m
LL	20.10kN /	5.20m =	3.87kN/m

Total dead UDL = **34.72kN/m**

Total Live UDL = **10.15kN/m**

Line 2 (middle)**Wall (300mm solid wall)**

DL	6.00kN/m ² x	7.20m =	43.20kN/m
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B1 distributed load (Left)

DL	7.00kN /	7.10m =	0.99kN/m
LL	5.60kN /	7.10m =	0.79kN/m

B1 distributed load (Right)

DL	7.00kN /	7.10m =	0.99kN/m
LL	5.60kN /	7.10m =	0.79kN/m

B2 distributed load (Left)

DL	12.30kN /	7.10m =	1.73kN/m
LL	18.30kN /	7.10m =	2.58kN/m

B2 distributed load (Right)

DL	12.30kN /	7.10m =	1.73kN/m
LL	18.30kN /	7.10m =	2.58kN/m

B2 distributed load (Left)

DL	12.30kN /	7.10m =	1.73kN/m
LL	18.30kN /	7.10m =	2.58kN/m

B2 distributed load (Right)

DL	12.30kN /	7.10m =	1.73kN/m
LL	18.30kN /	7.10m =	2.58kN/m

B3 distributed load (Left)

DL	6.00kN /	5.20m =	1.15kN/m
LL	18.90kN /	5.20m =	3.63kN/m

B3 distributed load (Right)

DL	6.00kN /	5.20m =	1.15kN/m
LL	18.90kN /	5.20m =	3.63kN/m

Total dead UDL	=	54.41kN/m
Total Live UDL	=	19.16kN/m

Line 3**Wall (cavity wall)**

DL	4.00kN/m ² x	6.50m =	26.00kN/m
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Floor

DL	0.50kN/m ² x	2.05m =	1.03kN/m
LL	2.00kN/m ² x	2.05m =	4.10kN/m

Tie joists (Attic)

DL	0.35kN/m ² x	0.60m =	0.21kN/m
LL	0.75kN/m ² x	0.60m =	0.45kN/m

Pitched Roof

DL	0.85kN/m ² x	1.20m =	1.02kN/m
LL	0.75kN/m ² x	1.20m =	0.90kN/m

TR4 distributed load

DL	2.00kN /	4.30m =	0.47kN/m
LL	3.80kN /	4.30m =	0.88kN/m

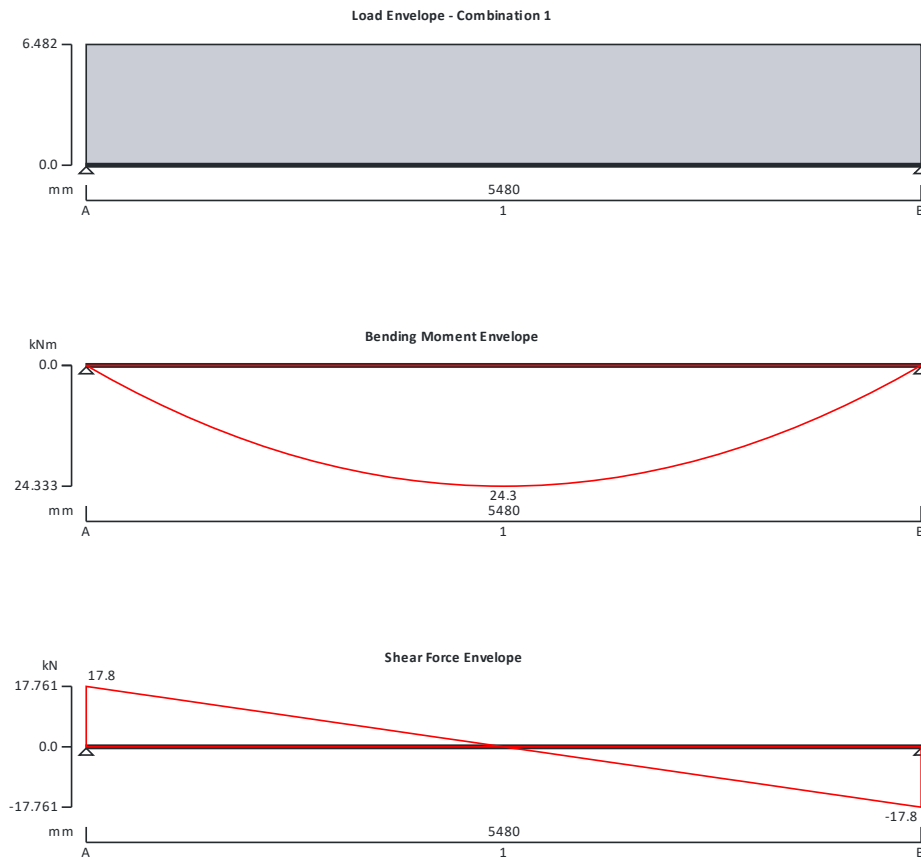
Total dead UDL	=	28.72kN/m
Total Live UDL	=	6.33kN/m

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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 3.0.18



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 2.3 kN/m Variable full UDL 2.03 kN/m
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Load combinations

Load combination 1	Support A	Permanent $\times 1.35$ Variable $\times 1.50$ Permanent $\times 1.35$
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Analysis results

Maximum moment

$M_{max} = 24.3 \text{ kNm}$

Variable $\times 1.50$

Maximum shear

$V_{max} = 17.8 \text{ kN}$

Permanent $\times 1.35$

Deflection

$\delta_{max} = 10.9 \text{ mm}$

Variable $\times 1.50$

Maximum reaction at support A

$R_{A_{max}} = 17.8 \text{ kN}$

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

Unfactored permanent load reaction at support A

$R_{A_{Permanent}} = 7 \text{ kN}$

$V_{min} = -17.8 \text{ kN}$

Unfactored variable load reaction at support A

$R_{A_{Variable}} = 5.6 \text{ kN}$

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

Maximum reaction at support B

$R_{B_{max}} = 17.8 \text{ kN}$

$R_{A_{min}} = 17.8 \text{ kN}$

Unfactored permanent load reaction at support B

$R_{B_{Permanent}} = 7 \text{ kN}$

$R_{B_{min}} = 17.8 \text{ kN}$

Unfactored variable load reaction at support B

$R_{B_{Variable}} = 5.6 \text{ kN}$

Section details

Section type

UB 203x133x25 (British Steel Section Range 2022 (BS4-1))

Steel grade

S275

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

Nominal thickness of element

$t = \max(t_f, t_w) = 7.8 \text{ mm}$

Nominal yield strength

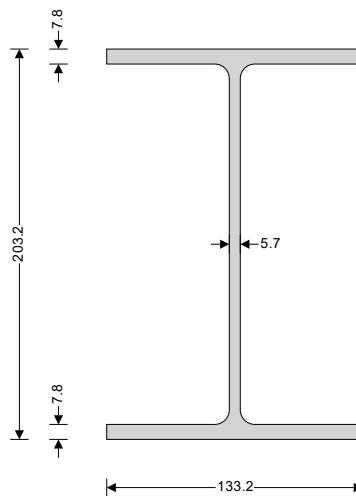
$f_y = 275 \text{ N/mm}^2$

Nominal ultimate tensile strength

$f_u = 410 \text{ N/mm}^2$

Modulus of elasticity

$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

Resistance of cross-sections

$\gamma_{M0} = 1.00$

Resistance of members to instability

$\gamma_{M1} = 1.00$

Resistance of tensile members to fracture

$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis

$K_y = 1.000$

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Effective length factor in minor axis

$$K_z = 1.000$$

Effective length factor for torsion

$$K_{LT,A} = 1.000$$

$$K_{LT,B} = 1.000$$

Classification of cross sections - Section 5.5

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

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

Width of section

$$c = d = 172.4 \text{ mm}$$

$$c / t_w = 32.7 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = 56.1 \text{ mm}$$

$$c / t_f = 7.8 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm}$$

Shear area factor

$$\eta = 1.000$$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 17.8 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1282 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 203.5 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

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

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 24.3 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 70.9 \text{ kNm}$$

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

$$g = \sqrt{[1 - (I_z / I_y)]} = 0.932$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

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

Unrestrained length

$$L = 1.0 \times L_{s1} = 5480 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 46.3 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.237$$

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

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.217$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.558$$

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Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.981}$

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.568}$

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{ply} \times f_y / \gamma_{M1} = \mathbf{40.3 \text{ kNm}}$

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 $\delta_{lim} = L_{s1} / 360 = \mathbf{15.2 \text{ mm}}$

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{10.934 \text{ mm}}$

PASS - Maximum deflection does not exceed deflection limit

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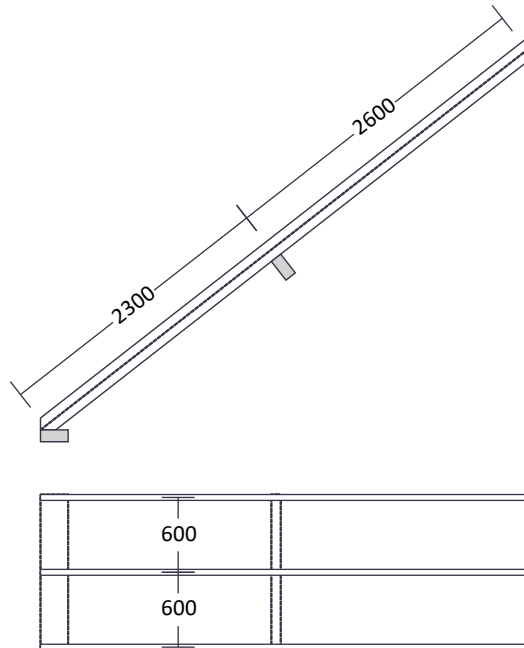
TIMBER RAFTER ANALYSIS & DESIGN (EN1995-1-1:2004)

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

Tedds calculation version 1.0.09

Rafter details

Description	47 x 150 C24 timber rafters
Rafter spacing	$S_{\text{Rafter}} = 600 \text{ mm}$
Rafter inclination	$\theta_{\text{Rafter}} = 38 \text{ deg}$



Forces input on Rafter

Permanent load on slope	$F_{G_Rafter} = 0.85 \text{ kN/m}^2$
Imposed load on plan	$F_{Q_Rafter} = 0.75 \text{ kN/m}^2$

Rafter loading details

Distributed loads

Permanent load on slope	$p_G = F_{G_Rafter} \times S_{\text{Rafter}} = 0.51 \text{ kN/m}$
Imposed load on slope	$p_Q = F_{Q_Rafter} \times S_{\text{Rafter}} \times \cos(\theta_{\text{Rafter}}) = 0.35 \text{ kN/m}$

Member span 1 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	9.24	0.539	0.058	PASS
Bearing stress	N/mm ²	1.1	0.636	0.578	PASS
Bending stress	N/mm ²	10.56	4.198	0.397	PASS
Shear stress	N/mm ²	1.76	0.465	0.264	PASS
Bending and axial force				0.401	PASS
Column stability check				0.469	PASS
Deflection	mm	9.2	1.3	0.141	PASS

		Project		Job Ref.	
		1a Blaendowlais, Dowlais, Merthyr Tydfil, CF48 3RB		AUX843	
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Member span 2 results summary	Unit	Capacity	Maximum	Utilisation	Result
Compressive stress	N/mm ²	9.24	0.286	0.031	PASS
Bearing stress	N/mm ²	1.1	0.404	0.367	PASS
Bending stress	N/mm ²	10.56	4.198	0.397	PASS
Shear stress	N/mm ²	1.76	0.5	0.284	PASS
Bending and axial force				0.398	PASS
Column stability check				0.439	PASS
Deflection	mm	10.4	2.761	0.265	PASS

ANALYSIS

Tedds calculation version 1.0.40

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

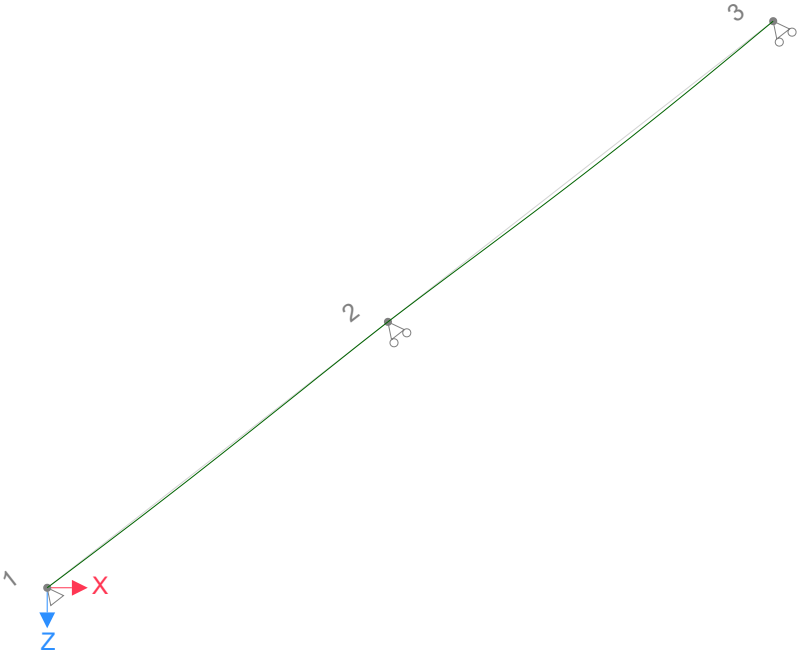
Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.51 kN/m at 0 m to 4.9 m
Member	Imposed	UDL	GlobalZ	0.35 kN/m at 0 m to 4.9 m

AUXILIUM STRUCTURAL ENGINEERING		Project 1a Blaendowlais, Dowlais, Merthyr Tydfil, CF48 3RB		Job Ref. AUX843	
		Section 2 Rafters		Sheet no./rev. 02/ 3	
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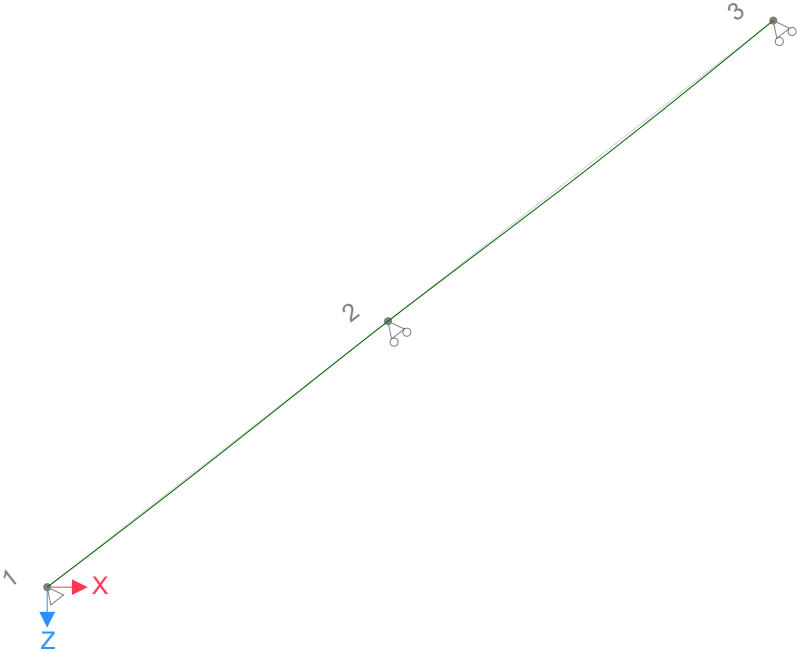
Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection



	Project 1a Blaendowlais, Dowlais, Merthyr Tydfil, CF48 3RB				Job Ref. AUX843	
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Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.09108	Member
2	-0.1	0	0.02976	Member
3	-0.1	0	-0.16413	Member

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.06462	Member
2	-0.1	0	0.02111	Member
3	-0.1	0	-0.11644	Member

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	6.2
1.00G + 1.00Q (Service)	0	4.4

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	2.3	1	-3.8	-0.8	0
		2	2	-1.5	-0.7
2	2.6	2	-2	-1.6	0.7
		3	0	-1	0

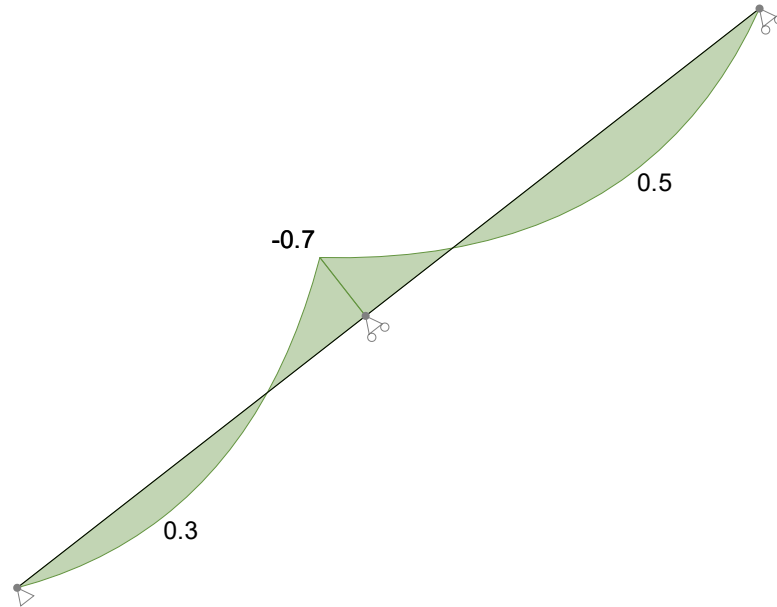
Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2.3	1	-2.7	-0.6	0
		2	1.4	-1	-0.5
2	2.6	2	-1.4	-1.1	0.5
		3	0	-0.7	0

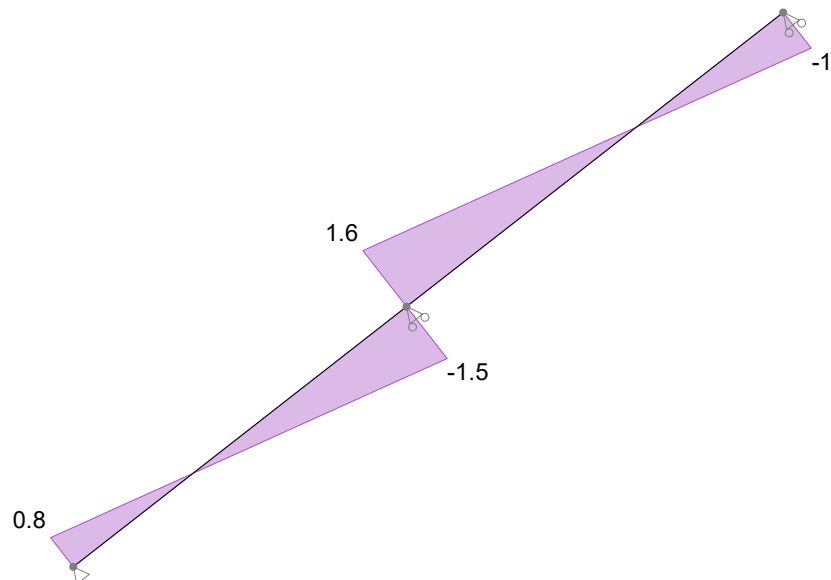
Project 1a Blaendowlais, Dowlais, Merthyr Tydfil, CF48 3RB				Job Ref. AUX843	
Section 2 Rafters				Sheet no./rev. 02/ 5	
Calc. by MK	Date 24-08-2026	Chk'd by MK	Date 24-08-2026	App'd by MK	Date 24-08-2026

Forces

Strength combinations - Moment envelope (kNm)



Strength combinations - Shear envelope (kN)



Project 1a Blaendowlais, Dowlais, Merthyr Tydfil, CF48 3RB				Job Ref. AUX843	
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Member results

Envelope - Strength combinations

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	2.3	-1.5		-0.7 (min)	
	3.887	0		0.5 (max)	

Tedds calculation version 2.2.28

Member - Span 1

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 $\gamma_M = 2.000$

Member details

Load duration - cl.2.3.1.2 Medium-term

Service class - cl.2.3.1.3 2

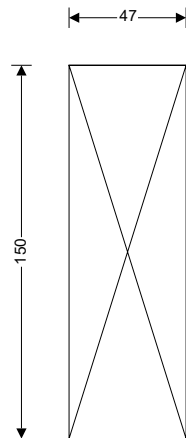
Timber section details

Number of timber sections in member N = 1

Breadth of sections b = 47 mm

Depth of sections h = 150 mm

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



47x150 timber section

Cross-sectional area, A, 7050 mm²

Section modulus, W_y, 176250 mm³

Section modulus, W_z, 55225 mm³

Second moment of area, I_y, 13218750 mm⁴

Second moment of area, I_z, 1297787 mm⁴

Radius of gyration, i_y, 43.3 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

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Check compression parallel to the grain - cl.6.1.4

Design axial compression $P_d = 3.8 \text{ kN}$
Design compressive stress $\sigma_{c,0,d} = P_d / A = 0.539 \text{ N/mm}^2$
Design compressive strength $f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c,0,k} / \gamma_M = 9.240 \text{ N/mm}^2$
 $\sigma_{c,0,d} / f_{c,0,d} = 0.058$

PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 3.885 \text{ kN}$
Effective contact length $L_{b,ef} = L_b + \min(L_b, 30 \text{ mm}) = 130 \text{ mm}$
Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.636 \text{ N/mm}^2$
Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.100 \text{ N/mm}^2$
 $\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.578$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 0.82 \text{ kN}$
Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.260 \text{ N/mm}^2$
Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 1.760 \text{ N/mm}^2$
 $\tau_{y,d} / f_{v,y,d} = 0.148$

PASS - Design shear strength exceeds design shear stress

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending $L_{e,y} = 0.9 \times 2300 \text{ mm} = 2070 \text{ mm}$
Slenderness ratio $\lambda_y = L_{e,y} / i_y = 47.805$
Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 0.811$
Effective length for z-axis bending $L_{e,z} = 0 \text{ mm}$
Slenderness ratio $\lambda_z = L_{e,z} / i_z = 0$
Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 0$

$\lambda_{rel,y} > 0.3$ column stability check is required

Straightness factor $\beta_c = 0.2$
Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 0.880$
 $k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 0.470$
 $k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = 0.819$
 $k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = 1.064$
Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) = 0.071$
 $\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) = 0.055$

PASS - Column stability is acceptable

Check design at end of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 3.038 \text{ kN}$
Effective contact length $L_{b,ef} = \min(L_b, 30 \text{ mm}) + L_b + \min(L_b, 30 \text{ mm}) = 160 \text{ mm}$
Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.404 \text{ N/mm}^2$

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Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.100 \text{ N/mm}^2$

$$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.367$$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 1.463 \text{ kN}$

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.465 \text{ N/mm}^2$

Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 1.760 \text{ N/mm}^2$

$$\tau_{y,d} / f_{v,y,d} = 0.264$$

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 0.74 \text{ kNm}$

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 4.198 \text{ N/mm}^2$

Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 10.56 \text{ N/mm}^2$

$$\sigma_{m,y,d} / f_{m,y,d} = 0.397$$

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} = 0.401$

$$(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d} = 0.282$$

PASS - Combined bending and axial compression utilisation is acceptable

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending $L_{e,y} = 0.9 \times 2300 \text{ mm} = 2070 \text{ mm}$

Slenderness ratio $\lambda_y = L_{e,y} / i_y = 47.805$

Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 0.811$

Effective length for z-axis bending $L_{e,z} = 0 \text{ mm}$

Slenderness ratio $\lambda_z = L_{e,z} / i_z = 0$

Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{(f_{c,0,k} / E_{0.05})} = 0$

$\lambda_{rel,y} > 0.3$ column stability check is required

Straightness factor $\beta_c = 0.2$

Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 0.880$

$$k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 0.470$$

$$k_{c,y} = 1 / (k_y + \sqrt{(k_y^2 - \lambda_{rel,y}^2)}) = 0.819$$

$$k_{c,z} = 1 / (k_z + \sqrt{(k_z^2 - \lambda_{rel,z}^2)}) = 1.064$$

Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} = 0.469$

$$\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} = 0.333$$

PASS - Column stability is acceptable

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

Check design 946 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection $\delta_y = 0.7 \text{ mm}$

Quasi-permanent variable load factor $\psi_2 = 0.3$

Final deflection with creep $\delta_{y,Final} = \delta_y \times (1 + k_{def}) = 1.3 \text{ mm}$

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Allowable deflection

$$\delta_{y, \text{Allowable}} = L_{m1_s1} / 250 = \mathbf{9.2 \text{ mm}}$$

$$\delta_{y, \text{Final}} / \delta_{y, \text{Allowable}} = \mathbf{0.141}$$

PASS - Allowable deflection exceeds final deflection

Member - Span 2

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3 $\gamma_M = \mathbf{2.000}$

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

$N = \mathbf{1}$

Breadth of sections

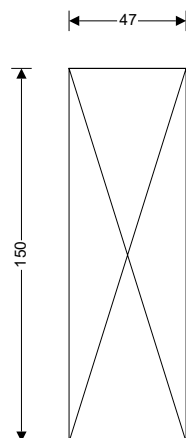
$b = \mathbf{47 \text{ mm}}$

Depth of sections

$h = \mathbf{150 \text{ mm}}$

Timber strength class - EN 338:2016 Table 1

C24



47x150 timber section

Cross-sectional area, A , 7050 mm²

Section modulus, W_y , 176250 mm³

Section modulus, W_z , 55225 mm³

Second moment of area, I_y , 13218750 mm⁴

Second moment of area, I_z , 1297787 mm⁴

Radius of gyration, i_y , 43.3 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

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

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

Modification factors

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

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

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

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

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

Check compression parallel to the grain - cl.6.1.4

Design axial compression

$P_d = \mathbf{2.016 \text{ kN}}$

Design compressive stress

$\sigma_{c,0,d} = P_d / A = \mathbf{0.286 \text{ N/mm}^2}$

Design compressive strength

$f_{c,0,d} = k_{mod} \times k_{sys} \times f_{c,0,k} / \gamma_M = \mathbf{9.240 \text{ N/mm}^2}$

$\sigma_{c,0,d} / f_{c,0,d} = \mathbf{0.031}$

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PASS - Design parallel compression strength exceeds design parallel compression stress

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 3.038 \text{ kN}$
Effective contact length $L_{b,ef} = \min(L_b, 30 \text{ mm}) + L_b + \min(L_b, 30 \text{ mm}) = 160 \text{ mm}$
Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.404 \text{ N/mm}^2$
Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.100 \text{ N/mm}^2$
 $\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.367$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 1.575 \text{ kN}$
Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.500 \text{ N/mm}^2$
Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 1.760 \text{ N/mm}^2$
 $\tau_{y,d} / f_{v,y,d} = 0.284$

PASS - Design shear strength exceeds design shear stress

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 0.74 \text{ kNm}$
Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 4.198 \text{ N/mm}^2$
Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 10.56 \text{ N/mm}^2$
 $\sigma_{m,y,d} / f_{m,y,d} = 0.397$

PASS - Design bending strength exceeds design bending stress

Check combined bending and axial compression - Section 6.2.4

Combined loading checks - exp.6.19 & 6.20 $(\sigma_{c,0,d} / f_{c,0,d})^2 + \sigma_{m,y,d} / f_{m,y,d} = 0.398$
 $(\sigma_{c,0,d} / f_{c,0,d})^2 + k_m \times \sigma_{m,y,d} / f_{m,y,d} = 0.279$

PASS - Combined bending and axial compression utilisation is acceptable

Check columns subjected to either compression or combined compression and bending - cl.6.3.2

Effective length for y-axis bending $L_{e,y} = 0.9 \times 2600 \text{ mm} = 2340 \text{ mm}$
Slenderness ratio $\lambda_y = L_{e,y} / i_y = 54.04$
Relative slenderness ratio - exp. 6.21 $\lambda_{rel,y} = \lambda_y / \pi \times \sqrt{f_{c,0,k} / E_{0.05}} = 0.916$
Effective length for z-axis bending $L_{e,z} = 0 \text{ mm}$
Slenderness ratio $\lambda_z = L_{e,z} / i_z = 0$
Relative slenderness ratio - exp. 6.22 $\lambda_{rel,z} = \lambda_z / \pi \times \sqrt{f_{c,0,k} / E_{0.05}} = 0$

$\lambda_{rel,y} > 0.3$ column stability check is required

Straightness factor $\beta_c = 0.2$
Instability factors - exp.6.25, 6.26, 6.27 & 6.28 $k_y = 0.5 \times (1 + \beta_c \times (\lambda_{rel,y} - 0.3) + \lambda_{rel,y}^2) = 0.981$
 $k_z = 0.5 \times (1 + \beta_c \times (\lambda_{rel,z} - 0.3) + \lambda_{rel,z}^2) = 0.470$
 $k_{c,y} = 1 / (k_y + \sqrt{k_y^2 - \lambda_{rel,y}^2}) = 0.750$
 $k_{c,z} = 1 / (k_z + \sqrt{k_z^2 - \lambda_{rel,z}^2}) = 1.064$
Column stability checks - exp.6.23 & 6.24 $\sigma_{c,0,d} / (k_{c,y} \times f_{c,0,d}) + \sigma_{m,y,d} / f_{m,y,d} = 0.439$
 $\sigma_{c,0,d} / (k_{c,z} \times f_{c,0,d}) + k_m \times \sigma_{m,y,d} / f_{m,y,d} = 0.307$

PASS - Column stability is acceptable

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

Check design 1453 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$$\delta_y = 1.5 \text{ mm}$$

Quasi-permanent variable load factor

$$\psi_2 = 0.3$$

Final deflection with creep

$$\delta_{y, \text{Final}} = \delta_y \times (1 + k_{\text{def}}) = 2.8 \text{ mm}$$

Allowable deflection

$$\delta_{y, \text{Allowable}} = L_{m1_s2} / 250 = 10.4 \text{ mm}$$

$$\delta_{y, \text{Final}} / \delta_{y, \text{Allowable}} = 0.265$$

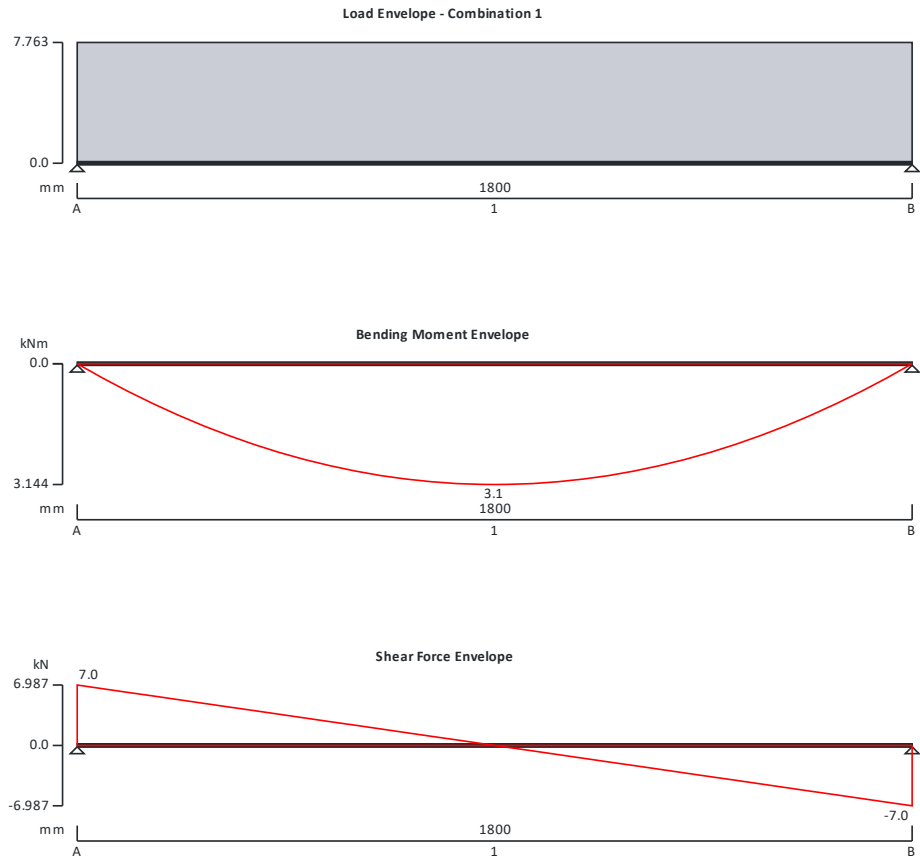
PASS - Allowable deflection exceeds final deflection

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

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

Tedds calculation version 1.7.05



Applied loading

Beam loads

Permanent self weight of beam × 1
Permanent full UDL 0.850 kN/m
Variable full UDL 3.400 kN/m
Permanent full UDL 1.050 kN/m

Load combinations

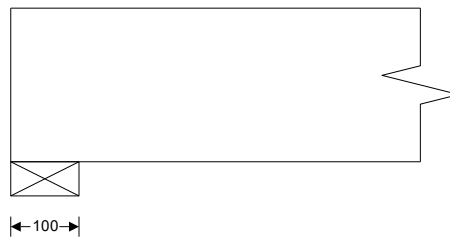
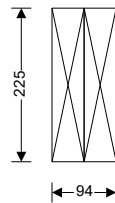
Load combination 1

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

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Analysis results

Maximum moment	$M_{\max} = 3.144 \text{ kNm}$	$M_{\min} = 0.000 \text{ kNm}$
Design moment	$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 3.144 \text{ kNm}$	
Maximum shear	$F_{\max} = 6.987 \text{ kN}$	$F_{\min} = -6.987 \text{ kN}$
Design shear	$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 6.987 \text{ kN}$	
Total load on beam	$W_{\text{tot}} = 13.973 \text{ kN}$	
Reactions at support A	$R_{A_{\max}} = 6.987 \text{ kN}$	$R_{A_{\min}} = 6.987 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 1.775 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 3.060 \text{ kN}$	
Reactions at support B	$R_{B_{\max}} = 6.987 \text{ kN}$	$R_{B_{\min}} = 6.987 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 1.775 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 3.060 \text{ kN}$	



Timber section details

Breadth of timber sections	$b = 47 \text{ mm}$
Depth of timber sections	$h = 225 \text{ mm}$
Number of timber sections in member	$N = 2$
Overall breadth of timber member	$b_b = N \times b = 94 \text{ mm}$
Timber strength class - EN 338:2016 Table 1	C24

Member details

Load duration - cl.2.3.1.2	Long-term
Service class of timber - cl.2.3.1.3	1
Length of span	$L_{s1} = 1800 \text{ mm}$
Length of bearing	$L_b = 100 \text{ mm}$

Section properties

Cross sectional area of member	$A = N \times b \times h = 21150 \text{ mm}^2$
Section modulus	$W_y = N \times b \times h^2 / 6 = 793125 \text{ mm}^3$
	$W_z = h \times (N \times b)^2 / 6 = 331350 \text{ mm}^3$
Second moment of area	$I_y = N \times b \times h^3 / 12 = 89226562 \text{ mm}^4$
	$I_z = h \times (N \times b)^3 / 12 = 15573450 \text{ mm}^4$
Radius of gyration	$r_y = \sqrt{I_y / A} = 65.0 \text{ mm}$
	$r_z = \sqrt{I_z / A} = 27.1 \text{ mm}$

Partial factor for material properties and resistances

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

Modification factor for load duration and moisture content - Table 3.1	
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Deformation factor for service classes - Table 3.2	$k_{mod} = 0.700$
Depth factor for bending - exp.3.1	$k_{def} = 0.600$
Depth factor for tension - exp.3.1	$k_{h,m} = 1.000$
Bending stress re-distribution factor - cl.6.1.6(2)	$k_{h,t} = 1.000$
Crack factor for shear resistance - cl.6.1.7(2)	$k_m = 0.700$
Load configuration factor - exp.6.4	$k_{cr} = 0.670$
System strength factor - cl.6.6	$k_{c,90} = 1.000$
Lateral buckling factor - cl.6.3.3(5)	$k_{sys} = 1.000$
	$k_{crit} = 1.000$

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress	$\sigma_{c,90,d} = R_{A_max} / (N \times b \times L_b) = 0.743 \text{ N/mm}^2$
Design compressive strength	$f_{c,90,d} = k_{mod} \times k_{sys} \times k_{c,90} \times f_{c,90,k} / \gamma_M = 1.346 \text{ N/mm}^2$
	$\sigma_{c,90,d} / f_{c,90,d} = 0.552$

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress	$\sigma_{m,d} = M / W_y = 3.964 \text{ N/mm}^2$
Design bending strength	$f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = 12.923 \text{ N/mm}^2$
	$\sigma_{m,d} / f_{m,d} = 0.307$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress	$\tau_d = 3 \times F / (2 \times k_{cr} \times A) = 0.740 \text{ N/mm}^2$
Permissible shear stress	$f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.154 \text{ N/mm}^2$
	$\tau_d / f_{v,d} = 0.343$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit	$\delta_{lim} = \min(14 \text{ mm}, 0.004 \times L_{s1}) = 7.200 \text{ mm}$
Instantaneous deflection due to permanent load	$\delta_{instG} = 0.340 \text{ mm}$
Final deflection due to permanent load	$\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = 0.545 \text{ mm}$
Instantaneous deflection due to variable load	$\delta_{instQ} = 0.587 \text{ mm}$
Factor for quasi-permanent variable action	$\psi_2 = 0.3$
Final deflection due to variable load	$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = 0.692 \text{ mm}$
Total final deflection	$\delta_{fin} = \delta_{finG} + \delta_{finQ} = 1.237 \text{ mm}$
	$\delta_{fin} / \delta_{lim} = 0.172$

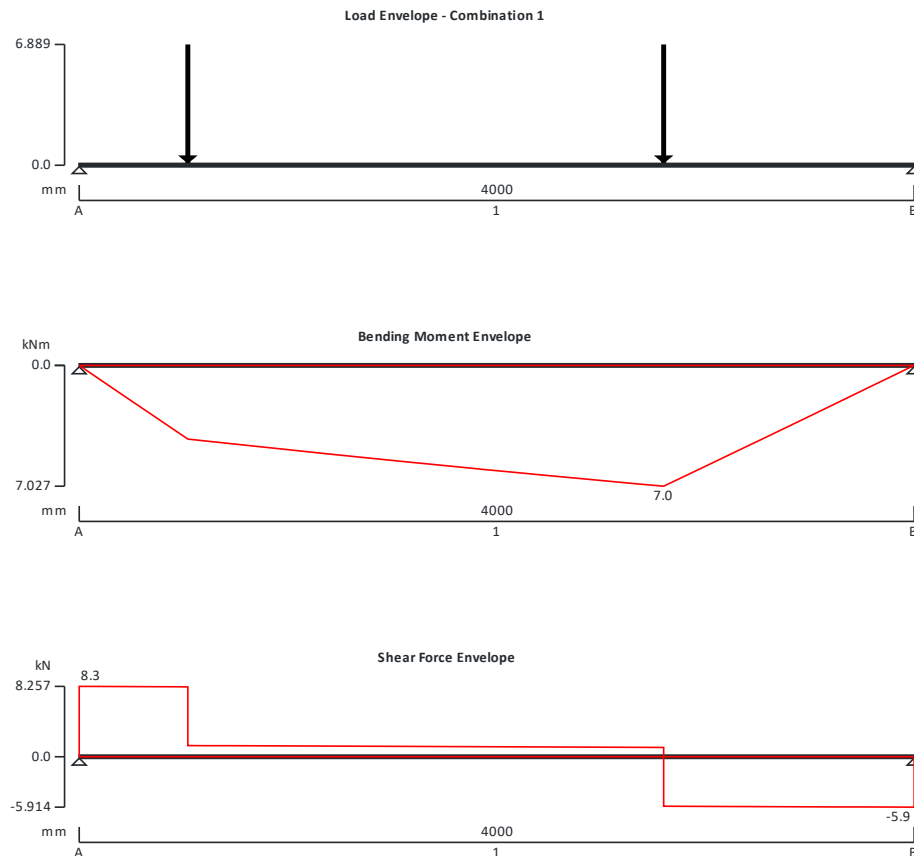
PASS - Total final deflection is less than the deflection limit

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

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

Tedds calculation version 1.7.05



Applied loading

Beam loads

Permanent self weight of beam $\times 1$
 Permanent point load 1.770 kN at 520 mm
 Variable point load 3.000 kN at 520 mm
 Permanent point load 1.770 kN at 2800 mm
 Variable point load 3.000 kN at 2800 mm

Load combinations

Load combination 1

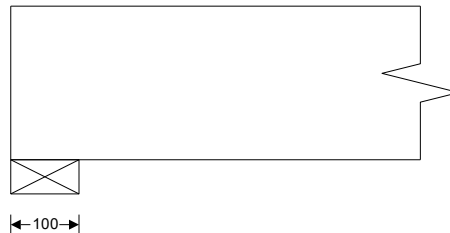
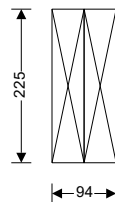
Support A	Permanent $\times 1.35$ Variable $\times 1.50$
Span 1	Permanent $\times 1.35$ Variable $\times 1.50$
Support B	Permanent $\times 1.35$

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Variable $\times 1.50$

Analysis results

Maximum moment	$M_{\max} = 7.027 \text{ kNm}$	$M_{\min} = 0.000 \text{ kNm}$
Design moment	$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 7.027 \text{ kNm}$	
Maximum shear	$F_{\max} = 8.257 \text{ kN}$	$F_{\min} = -5.914 \text{ kN}$
Design shear	$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 8.257 \text{ kN}$	
Total load on beam	$W_{\text{tot}} = 14.171 \text{ kN}$	
Reactions at support A	$R_{A_{\max}} = 8.257 \text{ kN}$	$R_{A_{\min}} = 8.257 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 2.216 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 3.510 \text{ kN}$	
Reactions at support B	$R_{B_{\max}} = 5.914 \text{ kN}$	$R_{B_{\min}} = 5.914 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 1.614 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 2.490 \text{ kN}$	



Timber section details

Breadth of timber sections	$b = 47 \text{ mm}$
Depth of timber sections	$h = 225 \text{ mm}$
Number of timber sections in member	$N = 2$
Overall breadth of timber member	$b_b = N \times b = 94 \text{ mm}$
Timber strength class - EN 338:2016 Table 1	C24

Member details

Load duration - cl.2.3.1.2	Long-term
Service class of timber - cl.2.3.1.3	1
Length of span	$L_{s1} = 4000 \text{ mm}$
Length of bearing	$L_b = 100 \text{ mm}$

Section properties

Cross sectional area of member	$A = N \times b \times h = 21150 \text{ mm}^2$
Section modulus	$W_y = N \times b \times h^2 / 6 = 793125 \text{ mm}^3$
	$W_z = h \times (N \times b)^2 / 6 = 331350 \text{ mm}^3$
Second moment of area	$I_y = N \times b \times h^3 / 12 = 89226562 \text{ mm}^4$
	$I_z = h \times (N \times b)^3 / 12 = 15573450 \text{ mm}^4$
Radius of gyration	$r_y = \sqrt{I_y / A} = 65.0 \text{ mm}$
	$r_z = \sqrt{I_z / A} = 27.1 \text{ mm}$

Partial factor for material properties and resistances

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

Modification factor for load duration and moisture content - Table 3.1

$$k_{mod} = \mathbf{0.700}$$

Deformation factor for service classes - Table 3.2

$$k_{def} = \mathbf{0.600}$$

Depth factor for bending - exp.3.1

$$k_{h,m} = \mathbf{1.000}$$

Depth factor for tension - exp.3.1

$$k_{h,t} = \mathbf{1.000}$$

Bending stress re-distribution factor - cl.6.1.6(2)

$$k_m = \mathbf{0.700}$$

Crack factor for shear resistance - cl.6.1.7(2)

$$k_{cr} = \mathbf{0.670}$$

Load configuration factor - exp.6.4

$$k_{c,90} = \mathbf{1.000}$$

System strength factor - cl.6.6

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

Lateral buckling factor - cl.6.3.3(5)

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

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress

$$\sigma_{c,90,d} = R_{A_max} / (N \times b \times L_b) = \mathbf{0.878 \text{ N/mm}^2}$$

Design compressive strength

$$f_{c,90,d} = k_{mod} \times k_{sys} \times k_{c,90} \times f_{c,90,k} / \gamma_M = \mathbf{1.346 \text{ N/mm}^2}$$

$$\sigma_{c,90,d} / f_{c,90,d} = \mathbf{0.653}$$

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress

$$\sigma_{m,d} = M / W_y = \mathbf{8.859 \text{ N/mm}^2}$$

Design bending strength

$$f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = \mathbf{12.923 \text{ N/mm}^2}$$

$$\sigma_{m,d} / f_{m,d} = \mathbf{0.686}$$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress

$$\tau_d = 3 \times F / (2 \times k_{cr} \times A) = \mathbf{0.874 \text{ N/mm}^2}$$

Permissible shear stress

$$f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = \mathbf{2.154 \text{ N/mm}^2}$$

$$\tau_d / f_{v,d} = \mathbf{0.406}$$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit

$$\delta_{lim} = \min(14 \text{ mm}, 0.004 \times L_{s1}) = \mathbf{14.000 \text{ mm}}$$

Instantaneous deflection due to permanent load

$$\delta_{instG} = \mathbf{3.225 \text{ mm}}$$

Final deflection due to permanent load

$$\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = \mathbf{5.159 \text{ mm}}$$

Instantaneous deflection due to variable load

$$\delta_{instQ} = \mathbf{5.031 \text{ mm}}$$

Factor for quasi-permanent variable action

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

Final deflection due to variable load

$$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = \mathbf{5.937 \text{ mm}}$$

Total final deflection

$$\delta_{fin} = \delta_{finG} + \delta_{finQ} = \mathbf{11.096 \text{ mm}}$$

$$\delta_{fin} / \delta_{lim} = \mathbf{0.793}$$

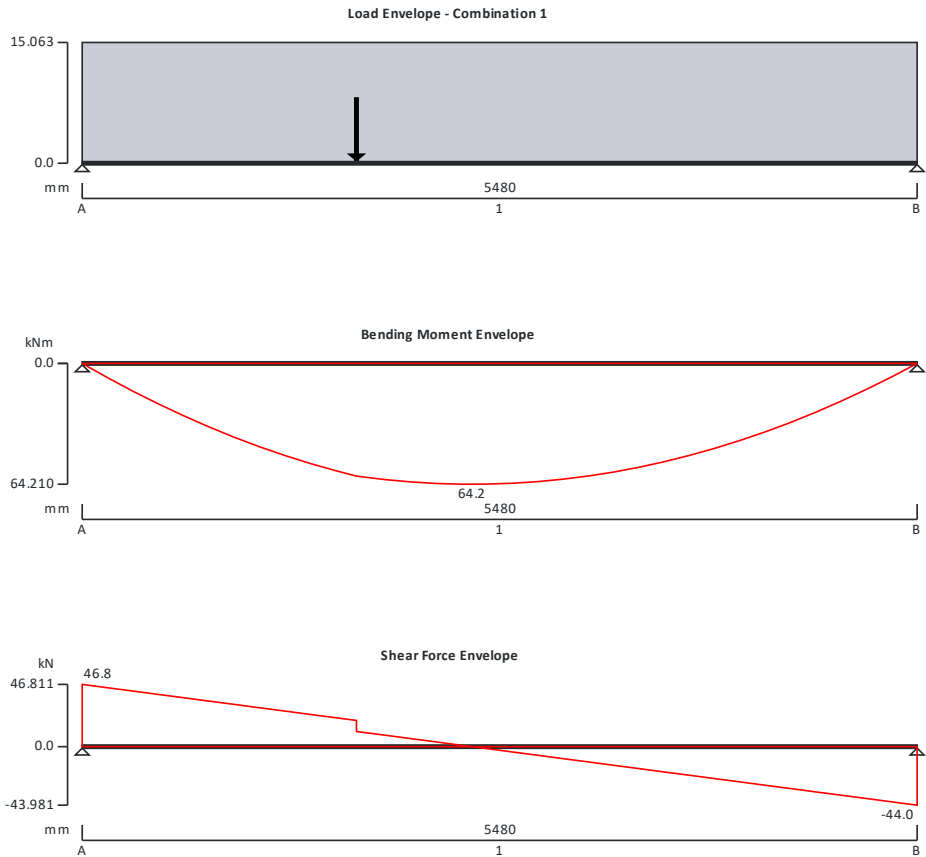
PASS - Total final deflection is less than the deflection limit

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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 3.0.18



Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1
	Permanent full UDL 3.85 kN/m
	Variable full UDL 6.25 kN/m
	Permanent point load 2.21 kN at 1800 mm
	Variable point load 3.51 kN at 1800 mm

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50

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Analysis results

Maximum moment

$M_{max} = 64.2$ kNm

Permanent $\times 1.35$

Maximum shear

$V_{max} = 46.8$ kN

Variable $\times 1.50$

Deflection

$\delta_{max} = 12$ mm

Permanent $\times 1.35$

Maximum reaction at support A

$R_{A_{max}} = 46.8$ kN

Variable $\times 1.50$

Unfactored permanent load reaction at support A

$R_{A_{Permanent}} = 13$ kN

$M_{min} = 0$ kNm

Unfactored variable load reaction at support A

$R_{A_{Variable}} = 19.5$ kN

$V_{min} = -44$ kN

Maximum reaction at support B

$R_{B_{max}} = 44$ kN

$\delta_{min} = 0$ mm

Unfactored permanent load reaction at support B

$R_{B_{Permanent}} = 12.3$ kN

$R_{A_{min}} = 46.8$ kN

Unfactored variable load reaction at support B

$R_{B_{Variable}} = 18.3$ kN

$R_{B_{min}} = 44$ kN

Section details

Section type

UB 254x146x37 (British Steel Section Range 2022 (BS4-1))

Steel grade

S275

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

Nominal thickness of element

$t = \max(t_r, t_w) = 10.9$ mm

Nominal yield strength

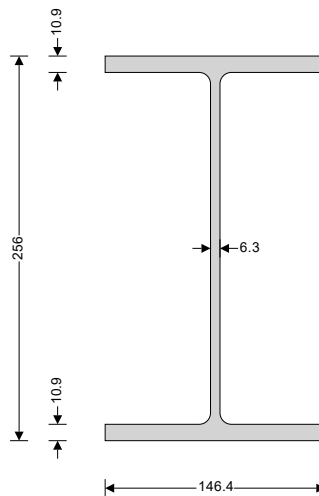
$f_y = 275$ N/mm²

Nominal ultimate tensile strength

$f_u = 410$ N/mm²

Modulus of elasticity

$E = 210000$ N/mm²



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

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Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section	$c = d = 219 \text{ mm}$	
	$c / t_w = 37.6 \times \varepsilon \leq 72 \times \varepsilon$	Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section	$c = (b - t_w - 2 \times r) / 2 = 62.5 \text{ mm}$	
	$c / t_f = 6.2 \times \varepsilon \leq 9 \times \varepsilon$	Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f = 234.2 \text{ mm}$
Shear area factor	$\eta = 1.000$
	$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 46.8 \text{ kN}$
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1759 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2)	$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 279.3 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

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

Design bending moment	$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 64.2 \text{ kNm}$
Design bending resistance moment - eq 6.13	$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 132.9 \text{ kNm}$

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	$g = \sqrt{[1 - (I_z / I_y)]} = 0.947$
Poissons ratio	$\nu = 0.3$
Shear modulus	$G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained length	$L = 1.0 \times L_{s1} = 5480 \text{ mm}$
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 101.4 \text{ kNm}$
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.145$
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	$\beta = 0.75$

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LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.118}$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.612}$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.977}$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.626}$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{ply} \times f_y / \gamma_{M1} = \mathbf{83.2 \text{ kNm}}$$

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

$$\delta_{lim} = L_{s1} / 360 = \mathbf{15.2 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{11.991 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

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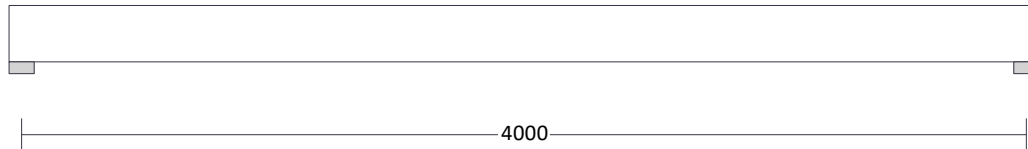
TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

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

Tedds calculation version 1.0.09

Joist details

Description 47 x 225 C24 timber joists
Joist spacing $S_{Joist} = 400$ mm



Forces input on Joist

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

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G_Joist} \times S_{Joist} = 0.20$ kN/m
Vertical imposed load on joist $p_Q = F_{Q_Joist} \times S_{Joist} = 0.80$ kN/m

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	1.692	0.5	0.296	PASS
Bending stress	N/mm ²	16.246	7.71	0.475	PASS
Shear stress	N/mm ²	2.708	0.647	0.239	PASS
Beam stability check				0.475	PASS
Deflection	mm	16	13.377	0.836	PASS

ANALYSIS

Tedds calculation version 1.0.40

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	0.2 kN/m at 0 m to 4 m
Member	Imposed	UDL	GlobalZ	0.8 kN/m at 0 m to 4 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	0.47598	
2	0	0	-0.47598	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.3249	
2	0	0	-0.3249	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	6.1
1.00G + 1.00Q (Service)	0	4.2

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	4	1	0	-3.1	0
		2	0	-3.1	0

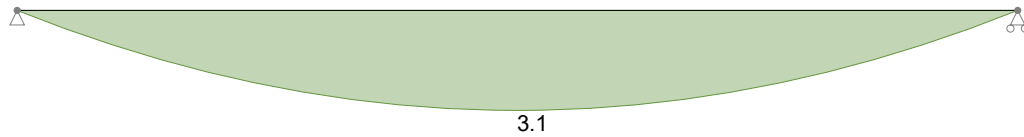
Project 1a Blaendowlais, Dowlais, Merthyr Tydfil, CF48 3RB				Job Ref. AUX843	
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Load combination: 1.00G + 1.00Q (Service)

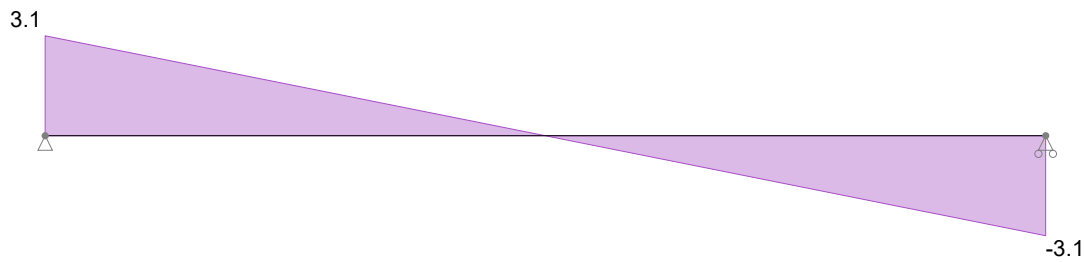
Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	4	1	0	-2.1	0
		2	0	-2.1	0

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	3.1 (max abs)		0 (min)	
	2	0		3.1 (max)	
	4	-3.1		0 (min)	

Tedds calculation version 2.2.28

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

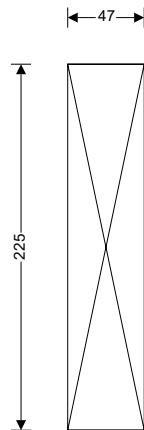
Number of timber sections in member $N = 1$

Breadth of sections $b = 47$ mm

Depth of sections $h = 225$ mm

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

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47x225 timber section

Cross-sectional area, A , 10575 mm²

Section modulus, W_y , 396562.5 mm³

Section modulus, W_z , 82838 mm³

Second moment of area, I_y , 44613281 mm⁴

Second moment of area, I_z , 1946681 mm⁴

Radius of gyration, i_y , 65 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 $F_{c,y,90,d} = 3.058$ kN

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

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.500$ 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

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.296$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 3.058$ kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.647$ N/mm²

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

$\tau_{y,d} / f_{v,y,d} = 0.239$

PASS - Design shear strength exceeds design shear stress

Check design 2000 mm along span

Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 3.058$ kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 7.71$ N/mm²

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Design bending strength

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

$$\sigma_{m,y,d} / f_{m,y,d} = \mathbf{0.475}$$

PASS - Design bending strength exceeds design bending stress

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

Lateral buckling factor - exp.6.34

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

Beam stability check - exp.6.33

$$\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}) = \mathbf{0.475}$$

PASS - Beam stability is acceptable

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

Check design 2000 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$$\delta_y = \mathbf{7.4 \text{ mm}}$$

Quasi-permanent variable load factor

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

Final deflection with creep

$$\delta_{y,Final} = \delta_y \times (1 + k_{def}) = \mathbf{13.4 \text{ mm}}$$

Allowable deflection

$$\delta_{y,Allowable} = L_{m1,s1} / 250 = \mathbf{16 \text{ mm}}$$

$$\delta_{y,Final} / \delta_{y,Allowable} = \mathbf{0.836}$$

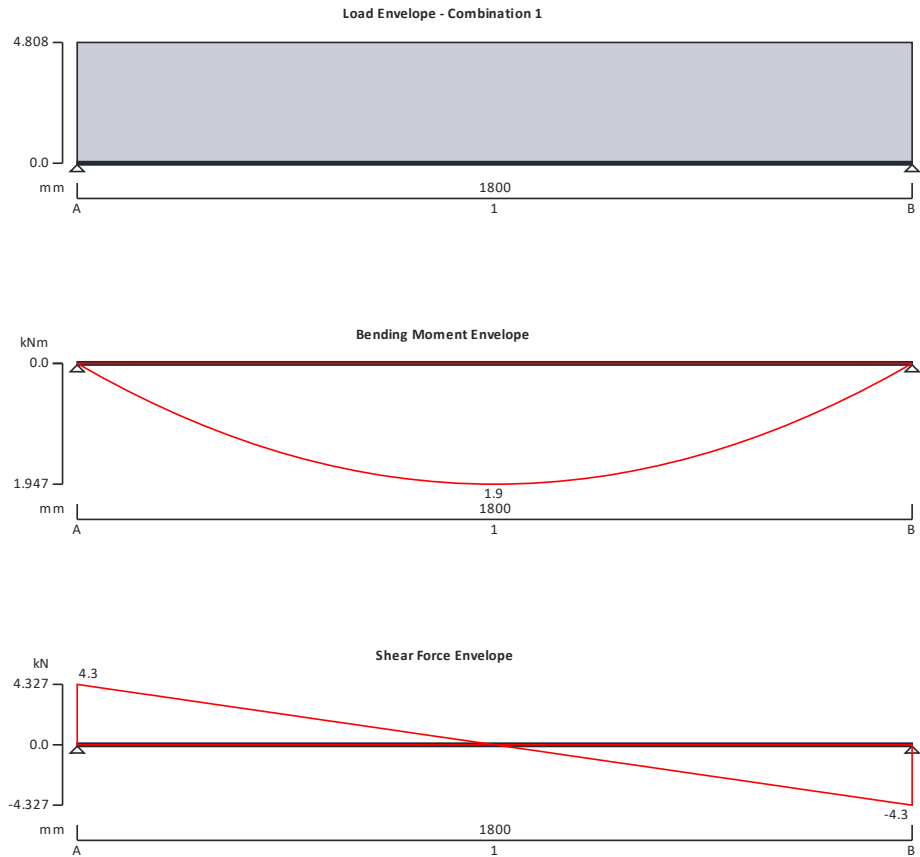
PASS - Allowable deflection exceeds final deflection

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

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

Tedds calculation version 1.7.05



Applied loading

Beam loads

Permanent self weight of beam × 1
Permanent full UDL 1.600 kN/m
Variable full UDL 1.700 kN/m

Load combinations

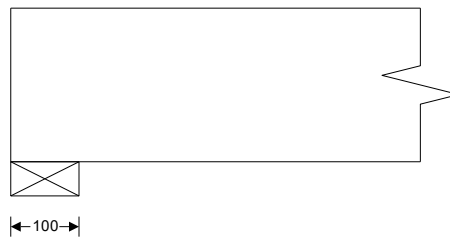
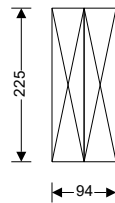
Load combination 1

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

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Analysis results

Maximum moment	$M_{\max} = 1.947 \text{ kNm}$	$M_{\min} = 0.000 \text{ kNm}$
Design moment	$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 1.947 \text{ kNm}$	
Maximum shear	$F_{\max} = 4.327 \text{ kN}$	$F_{\min} = -4.327 \text{ kN}$
Design shear	$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 4.327 \text{ kN}$	
Total load on beam	$W_{\text{tot}} = 8.654 \text{ kN}$	
Reactions at support A	$R_{A_{\max}} = 4.327 \text{ kN}$	$R_{A_{\min}} = 4.327 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 1.505 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 1.530 \text{ kN}$	
Reactions at support B	$R_{B_{\max}} = 4.327 \text{ kN}$	$R_{B_{\min}} = 4.327 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 1.505 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 1.530 \text{ kN}$	



Timber section details

Breadth of timber sections	$b = 47 \text{ mm}$
Depth of timber sections	$h = 225 \text{ mm}$
Number of timber sections in member	$N = 2$
Overall breadth of timber member	$b_b = N \times b = 94 \text{ mm}$
Timber strength class - EN 338:2016 Table 1	C24

Member details

Load duration - cl.2.3.1.2	Long-term
Service class of timber - cl.2.3.1.3	1
Length of span	$L_{s1} = 1800 \text{ mm}$
Length of bearing	$L_b = 100 \text{ mm}$

Section properties

Cross sectional area of member	$A = N \times b \times h = 21150 \text{ mm}^2$
Section modulus	$W_y = N \times b \times h^2 / 6 = 793125 \text{ mm}^3$
	$W_z = h \times (N \times b)^2 / 6 = 331350 \text{ mm}^3$
Second moment of area	$I_y = N \times b \times h^3 / 12 = 89226562 \text{ mm}^4$
	$I_z = h \times (N \times b)^3 / 12 = 15573450 \text{ mm}^4$
Radius of gyration	$r_y = \sqrt{I_y / A} = 65.0 \text{ mm}$
	$r_z = \sqrt{I_z / A} = 27.1 \text{ mm}$

Partial factor for material properties and resistances

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

Modification factor for load duration and moisture content - Table 3.1	
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Deformation factor for service classes - Table 3.2	$k_{mod} = 0.700$
Depth factor for bending - exp.3.1	$k_{def} = 0.600$
Depth factor for tension - exp.3.1	$k_{h,m} = 1.000$
Bending stress re-distribution factor - cl.6.1.6(2)	$k_{h,t} = 1.000$
Crack factor for shear resistance - cl.6.1.7(2)	$k_m = 0.700$
Load configuration factor - exp.6.4	$k_{cr} = 0.670$
System strength factor - cl.6.6	$k_{c,90} = 1.000$
Lateral buckling factor - cl.6.3.3(5)	$k_{sys} = 1.000$
	$k_{crit} = 1.000$

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress	$\sigma_{c,90,d} = R_{A_max} / (N \times b \times L_b) = 0.460 \text{ N/mm}^2$
Design compressive strength	$f_{c,90,d} = k_{mod} \times k_{sys} \times k_{c,90} \times f_{c,90,k} / \gamma_M = 1.346 \text{ N/mm}^2$
	$\sigma_{c,90,d} / f_{c,90,d} = 0.342$

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress	$\sigma_{m,d} = M / W_y = 2.455 \text{ N/mm}^2$
Design bending strength	$f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = 12.923 \text{ N/mm}^2$
	$\sigma_{m,d} / f_{m,d} = 0.190$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress	$\tau_d = 3 \times F / (2 \times k_{cr} \times A) = 0.458 \text{ N/mm}^2$
Permissible shear stress	$f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.154 \text{ N/mm}^2$
	$\tau_d / f_{v,d} = 0.213$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit	$\delta_{lim} = \min(14 \text{ mm}, 0.004 \times L_{s1}) = 7.200 \text{ mm}$
Instantaneous deflection due to permanent load	$\delta_{instG} = 0.289 \text{ mm}$
Final deflection due to permanent load	$\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = 0.462 \text{ mm}$
Instantaneous deflection due to variable load	$\delta_{instQ} = 0.293 \text{ mm}$
Factor for quasi-permanent variable action	$\psi_2 = 0.3$
Final deflection due to variable load	$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = 0.346 \text{ mm}$
Total final deflection	$\delta_{fin} = \delta_{finG} + \delta_{finQ} = 0.808 \text{ mm}$
	$\delta_{fin} / \delta_{lim} = 0.112$

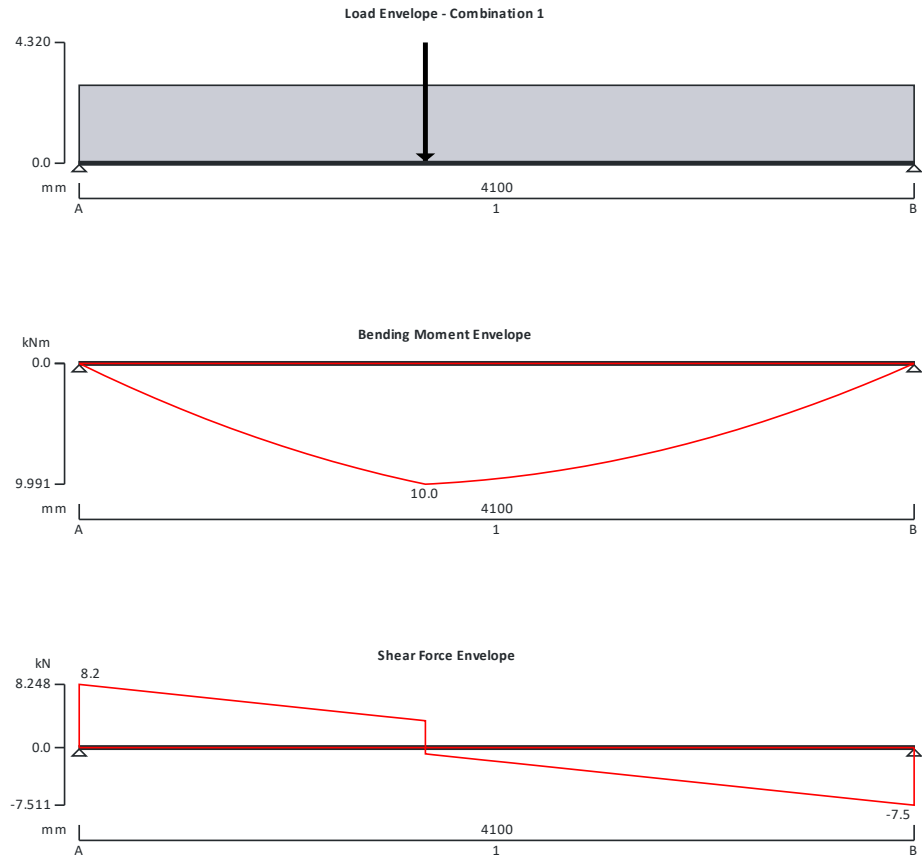
PASS - Total final deflection is less than the deflection limit

AUXILIUM STRUCTURAL ENGINEERING		Project 1a Blaendowlais, Dowlais, Merthyr Tydfil, CF48 3RB		Job Ref. AUX843	
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FLITCH BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

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

Tedds calculation version 1.7.05



Applied loading

Beam loads

- Permanent self weight of beam × 1
- Permanent full UDL 0.300 kN/m
- Variable full UDL 1.400 kN/m
- Permanent point load 1.500 kN at 1700 mm
- Variable point load 1.530 kN at 1700 mm

Load combinations

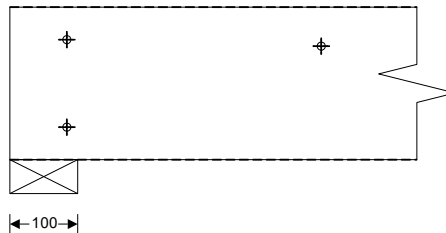
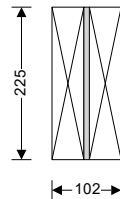
Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35

Project 1a Blaendowlais, Dowlais, Merthyr Tydfil, CF48 3RB				Job Ref. AUX843	
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Variable $\times 1.50$

Analysis results

Maximum moment	$M_{\max} = 9.991 \text{ kNm}$	$M_{\min} = 0.000 \text{ kNm}$
Design moment	$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 9.991 \text{ kNm}$	
Maximum shear	$F_{\max} = 8.248 \text{ kN}$	$F_{\min} = -7.511 \text{ kN}$
Design shear	$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 8.248 \text{ kN}$	
Total load on beam	$W_{\text{tot}} = 15.759 \text{ kN}$	
Reactions at support A	$R_{A_{\max}} = 8.248 \text{ kN}$	$R_{A_{\min}} = 8.248 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 1.926 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 3.766 \text{ kN}$	
Reactions at support B	$R_{B_{\max}} = 7.511 \text{ kN}$	$R_{B_{\min}} = 7.511 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 1.670 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 3.504 \text{ kN}$	



Timber section details

Breadth of timber sections	$b = 47 \text{ mm}$
Depth of timber sections	$h = 225 \text{ mm}$
Number of timber sections in member	$N = 2$
Timber strength class - EN 338:2016 Table 1	C24

Steel section details

Breadth of steel plate	$b_s = 8 \text{ mm}$
Depth of steel plate	$h_s = 225 \text{ mm}$
Number of steel plates in member	$N_s = 1$
Nominal yield stress	$f_y = 275 \text{ N/mm}^2$
Bolt diameter	$\phi_b = 12 \text{ mm}$

Member details

Load duration - cl.2.3.1.2	Long-term
Service class of timber - cl.2.3.1.3	1
Length of span	$L_{s1} = 4100 \text{ mm}$
Length of bearing	$L_b = 100 \text{ mm}$

Section properties

Cross sectional area of member	$A = N \times b \times h = 21150 \text{ mm}^2$
Timber section modulus	$W_{yt} = N \times b \times h^2 / 6 = 793125 \text{ mm}^3$
Steel section modulus	$W_{ys} = N_s \times b_s \times h_s^2 / 6 = 67500 \text{ mm}^3$
Second moment of area of timber	$I_{yt} = N \times b \times h^3 / 12 = 89226562 \text{ mm}^4$
Second moment of area of steel	$I_{ys} = N_s \times b_s \times h_s^3 / 12 = 7593750 \text{ mm}^4$

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Load proportions

Instant deflection under permanent actions $U_{instG} = 1.556$ mm

Instant deflection under principal variable action $U_{instQ1} = 2.860$ mm

$$k_{def} = 0.6$$

$$\psi_2 = 0.3$$

Final fifth percentile value of modulus of elasticity

$$E_{0.05.fin} = E_{0.05} \times (U_{instG} + U_{instQ1}) / (U_{instG} + U_{instQ1} + k_{def} \times (U_{instG} + \psi_2 \times U_{instQ1})) = 5572 \text{ N/mm}^2$$

Final mean value of modulus of elasticity

$$E_{0.mean.fin} = E_{0.mean} \times (U_{instG} + U_{instQ1}) / (U_{instG} + U_{instQ1} + k_{def} \times (U_{instG} + \psi_2 \times U_{instQ1})) = 8283 \text{ N/mm}^2$$

Proportion of applied load in timber to check bending, shear and instantaneous deflection

$$k_t = E_{0.mean} \times I_{yt} / (E_{0.mean} \times I_{yt} + E_{SEC3} \times I_{ys}) = 0.381$$

Proportion of applied load in steel to check strength of bolts and steel plate

$$k_s = 1.1 \times E_{SEC3} \times I_{ys} / (E_{0.05.fin} \times I_{yt} + E_{SEC3} \times I_{ys}) = 0.839$$

Proportion of applied load in timber to check final deflection

$$k_{t.def} = E_{0.mean.fin} \times I_{yt} / (E_{0.mean.fin} \times I_{yt} + E_{SEC3} \times I_{ys}) = 0.317$$

Partial factor for material properties and resistances

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

Modification factors

Modification factor for load duration and moisture content - Table 3.1

$$k_{mod} = 0.700$$

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

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

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

Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 1.000$

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

Load configuration factor - exp.6.4 $k_{c,90} = 1.000$

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

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

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress $\sigma_{c,90.d} = R_{A_max} / (N \times b \times L_b) = 0.877 \text{ N/mm}^2$

Design compressive strength $f_{c,90.d} = k_{mod} \times k_{sys} \times k_{c,90} \times f_{c,90,k} / \gamma_M = 1.346 \text{ N/mm}^2$

$$\sigma_{c,90.d} / f_{c,90.d} = 0.652$$

PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design timber bending stress $\sigma_{m,t.d} = k_t \times M / W_{yt} = 4.799 \text{ N/mm}^2$

Design timber bending strength $f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = 12.923 \text{ N/mm}^2$

$$\sigma_{m,t.d} / f_{m,d} = 0.371$$

PASS - Design timber bending strength exceeds design timber bending stress

Design steel bending stress $\sigma_{m,s.d} = k_s \times M / W_{ys} = 124.114 \text{ N/mm}^2$

Design steel bending strength $f_{y,d} = f_y / \gamma_{M0} = 275.000 \text{ N/mm}^2$

$$\sigma_{m,s.d} / f_{y,d} = 0.451$$

PASS - Design steel bending strength exceeds design steel bending stress

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Shear - cl.6.1.7

Applied shear stress

$$\tau_d = 3 \times k_t \times F / (2 \times k_{cr} \times A) = \mathbf{0.333 \text{ N/mm}^2}$$

Permissible shear stress

$$f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = \mathbf{2.154 \text{ N/mm}^2}$$

$$\tau_d / f_{v,d} = \mathbf{0.154}$$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit

$$\delta_{lim} = \min(14 \text{ mm}, 0.004 \times L_{s1}) = \mathbf{14.000 \text{ mm}}$$

Instantaneous deflection due to permanent load

$$\delta_{instG} = \mathbf{1.765 \text{ mm}}$$

Final deflection due to permanent load

$$\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = \mathbf{2.823 \text{ mm}}$$

Instantaneous deflection due to variable load

$$\delta_{instQ} = \mathbf{3.220 \text{ mm}}$$

Factor for quasi-permanent variable action

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

Final deflection due to variable load

$$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = \mathbf{3.800 \text{ mm}}$$

Total final deflection

$$\delta_{fin} = \delta_{finG} + \delta_{finQ} = \mathbf{6.623 \text{ mm}}$$

$$\delta_{fin} / \delta_{lim} = \mathbf{0.473}$$

PASS - Total final deflection is less than the deflection limit

Steel-to-timber connections - cl.8.2.3

Characteristic yield moment - exp.8.30

$$M_{y,Rk} = 0.3 \text{ mm}^{0.4} \times f_{u,k} \times \phi_b^{2.6} = \mathbf{76745 \text{ Nmm}}$$

Char.embed.strength par.to grain - exp.8.32

$$f_{h,0,k} = 0.082e9 \text{ m/sec}^2 \times (1 \text{ mm} - 0.01 \times \phi_b) \times \rho_k = \mathbf{25.256 \text{ N/mm}^2}$$

$$k_{90} = 1.35 + 0.015 \times \phi_b / 1 \text{ mm} = \mathbf{1.530}$$

Char.embed.strength perp.to grain - exp.8.31

$$f_{h,90,k} = f_{h,0,k} / k_{90} = \mathbf{16.507 \text{ N/mm}^2}$$

Thickness limit for thin steel plates

$$b_{s,thin} = \phi_b / 2 = \mathbf{6 \text{ mm}}$$

Thickness limit for thick steel plates

$$b_{s,thk} = \phi_b = \mathbf{12 \text{ mm}}$$

Characteristic load-carrying capacity for a plate of any thickness as the central member in double shear - exp.8.11

$$F_{v,Rk,f} = f_{h,k} \times b \times \phi_b = \mathbf{9.310 \text{ kN}}$$

$$F_{v,Rk,g} = f_{h,k} \times b \times \phi_b \times (\sqrt{[2 + 4 \times M_{y,Rk} / (f_{h,k} \times \phi_b \times b^2)]} - 1) = \mathbf{5.992 \text{ kN}}$$

$$F_{v,Rk,h} = 2.3 \times \sqrt{[M_{y,Rk} \times f_{h,k} \times \phi_b]} = \mathbf{8.968 \text{ kN}}$$

$$F_{v,Rk} = \min(F_{v,Rk,f}, F_{v,Rk,g}, F_{v,Rk,h}) = \mathbf{5.992 \text{ kN}}$$

Flitch plate bolting requirements

Total load on member

$$W_{tot} = \mathbf{15.759 \text{ kN}}$$

Total load taken by steel

$$W_s = k_s \times W_{tot} = \mathbf{13.215 \text{ kN}}$$

Number of interfaces

$$N_{int} = (N + N_s) - 1 = \mathbf{2}$$

Number of bolts required at supports

$$N_{be} = \max(k_s \times R_{A,max} / (N_{int} \times F_{v,Rk}), 2) = \mathbf{2}$$

Limiting bolt spacing

$$S_{limit} = \min(2.5 \times h, 600 \text{ mm}) = \mathbf{563 \text{ mm}}$$

Maximum bolt spacing

$$S_{max} = \mathbf{375 \text{ mm}}$$

Minimum number of bolts along length of member

$$N_{bl} = W_s / (N_{int} \times F_{v,Rk}) = \mathbf{1.103}$$

Provide a minimum of 2 No.12 mm diameter bolts at each support

Provide 12 mm diameter bolts at maximum 375 mm centres staggered 55 mm alternately above and below the centre line

Minimum bolt spacings - cl.8.5

Minimum end spacing - Table 8.4

$$S_{end} = 7 \times \phi_b = \mathbf{84 \text{ mm}}$$

Minimum edge spacing - Table 8.4

$$S_{edge} = 4 \times \phi_b = \mathbf{48 \text{ mm}}$$

Minimum bolt spacing - Table 8.4

$$S_{bolt} = 4 \times \phi_b = \mathbf{48 \text{ mm}}$$

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Minimum washer diameter - cl.10.4.3

$\phi_w = 3 \times \phi_b = \mathbf{36\text{ mm}}$

Minimum washer thickness - cl.10.4.3

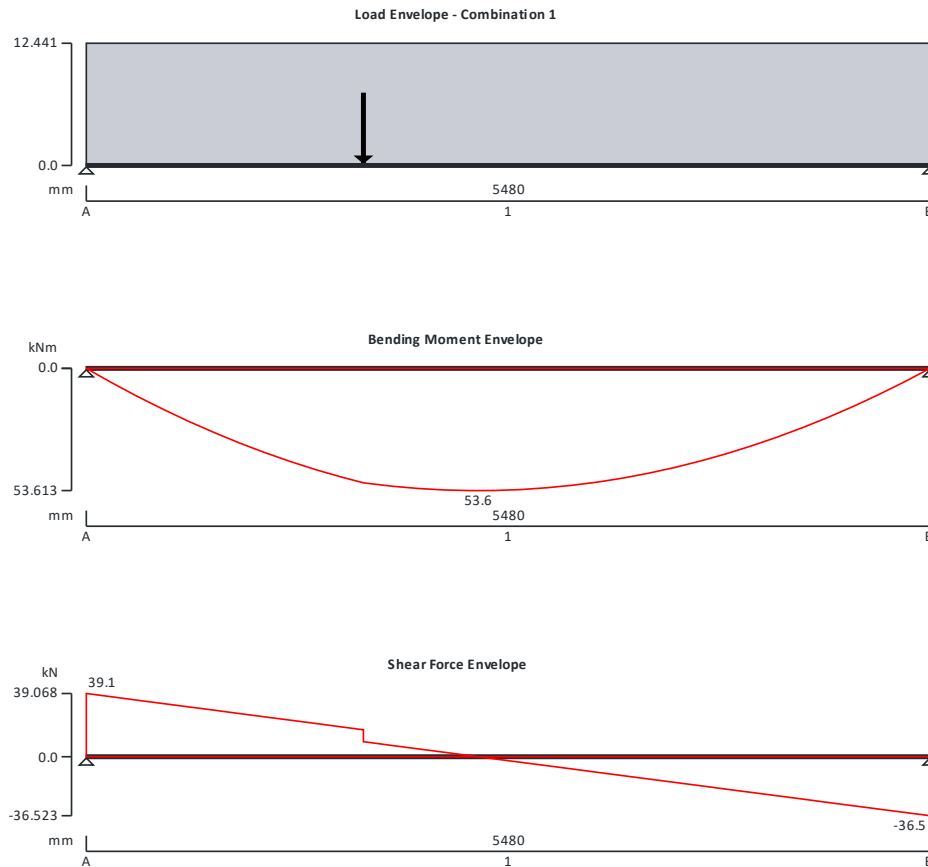
$t_w = 0.3 \times \phi_b = \mathbf{3.6\text{ mm}}$

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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 3.0.18



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 1.63 kN/m Variable full UDL 6.5 kN/m Permanent point load 1.66 kN at 1800 mm Variable point load 3.45 kN at 1800 mm
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Load combinations

Load combination 1	Support A	Permanent $\times 1.35$
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Support B

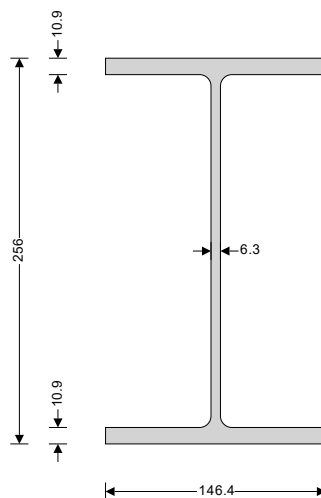
Variable $\times 1.50$
 Permanent $\times 1.35$
 Variable $\times 1.50$
 Permanent $\times 1.35$
 Variable $\times 1.50$

Analysis results

Maximum moment	$M_{\max} = 53.6 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 39.1 \text{ kN}$	$V_{\min} = -36.5 \text{ kN}$
Deflection	$\delta_{\max} = 9.9 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 39.1 \text{ kN}$	$R_{A_{\min}} = 39.1 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_{\text{Permanent}}} = 6.6 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_{\text{Variable}}} = 20.1 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 36.5 \text{ kN}$	$R_{B_{\min}} = 36.5 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_{\text{Permanent}}} = 6 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_{\text{Variable}}} = 18.9 \text{ kN}$	

Section details

Section type	UB 254x146x37 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_f, t_w) = 10.9 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1

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

Lateral restraint

Span 1 has lateral restraint at supports only

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Effective length factors

Effective length factor in major axis	$K_y = 1.000$
Effective length factor in minor axis	$K_z = 1.000$
Effective length factor for torsion	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section	$c = d = 219 \text{ mm}$	
	$c / t_w = 37.6 \times \varepsilon \leq 72 \times \varepsilon$	Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section	$c = (b - t_w - 2 \times r) / 2 = 62.5 \text{ mm}$	
	$c / t_f = 6.2 \times \varepsilon \leq 9 \times \varepsilon$	Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f = 234.2 \text{ mm}$
Shear area factor	$\eta = 1.000$
	$h_w / t_w < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force	$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 39.1 \text{ kN}$
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1759 \text{ mm}^2$
Design shear resistance - cl 6.2.6(2)	$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 279.3 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

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

Design bending moment	$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 53.6 \text{ kNm}$
Design bending resistance moment - eq 6.13	$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 132.9 \text{ kNm}$

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	$g = \sqrt{[1 - (I_z / I_y)]} = 0.947$
Poissons ratio	$\nu = 0.3$
Shear modulus	$G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$
Unrestrained length	$L = 1.0 \times L_{s1} = 5480 \text{ mm}$
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 101.4 \text{ kNm}$
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.145$
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	$\beta = 0.75$

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LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.118}$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.612}$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.977}$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.626}$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{ply} \times f_y / \gamma_{M1} = \mathbf{83.2 \text{ kNm}}$$

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

$$\delta_{lim} = L_{s1} / 360 = \mathbf{15.2 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{9.85 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

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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	17.9 kN	37.7 kN	0.55 N/mm ²	2.84 N/mm ²	0.193	Pass

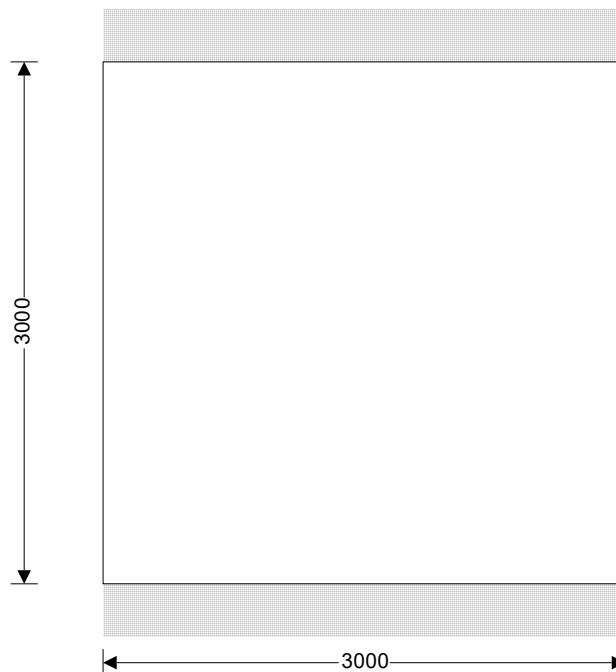
Masonry panel details

Panel length L = 3000 mm

Panel height h = 3000 mm

Panel support conditions

Top and bottom edges supported only



Effective height of masonry wall - Section 5.5.1.2

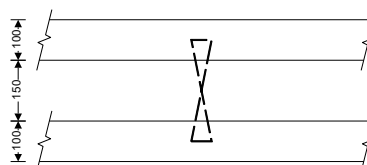
Vertical restraints Wall restrained at top and bottom only

Horizontal restraints **Simply supported**

Reduction factor - eq. 5.5 $\rho_2 = 1.00$

Effective height of wall - eq. 5.2 $h_{ef} = \rho_2 \times h = 3000 \text{ mm}$

Wall construction details



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Wall type

Cavity wall

Inner leaf thickness

$t_2 = 100.0$ mm

Cavity thickness

$t_c = 150.0$ 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

$k_{tef} = 1.00$

Effective thickness - eq. 5.11

$t_{ef} = (k_{tef} \times t_{ef1}^3 + t_{ef2}^3)^{1/3} = 126.0$ mm

Masonry material details

Unit type

Aggregate concrete - Group 1

Compressive strength of masonry unit

$f_c = 7.3$ N/mm²

Height of unit

$h_u = 215$ mm

Width of unit

$w_u = 102$ mm

Conditioning factor

$k = 1.0$

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

Shape factor - Table A.1

$d_{sf} = 1.376$

Mean compressive strength of masonry unit

$f_b = f_c \times k \times d_{sf} = 10.04$ N/mm²

Specific weight of units

$\gamma = 18$ kN/m³

Mortar type

M4 - General Purpose

Compressive strength of mortar

$f_m = 4.0$ N/mm²

Compressive strength factor - Tbl. NA 4

$K = 0.75$

Characteristic compressive strength - eq. 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3} = 5.72$ N/mm²

Short term secant modulus of elasticity factor

$K_E = 1000$

Modulus of elasticity - cl.3.7.2

$E_w = K_E \times f_k = 5715$ N/mm²

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

$A = L \times t = 0.30$ m²

Design compressive strength of masonry

$f_d = f_k / \gamma_M = 1.91$ N/mm²

Partial safety factors for design loads

Partial safety factor for permanent load

$\gamma_{FG} = 1.35$

Partial safety factor for variable load

$\gamma_{FQ} = 1.50$

Superimposed vertical loading details

Permanent UDL at top of wall

$g_k = 0.00$ kN/m

Variable UDL at top of wall

$q_k = 0.00$ kN/m

Eccentricity of permanent UDL load

$e_{gu} = 0$ mm

Eccentricity of variable UDL load

$e_{qu} = 0$ mm

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit

$\lambda_{lim} = 27$

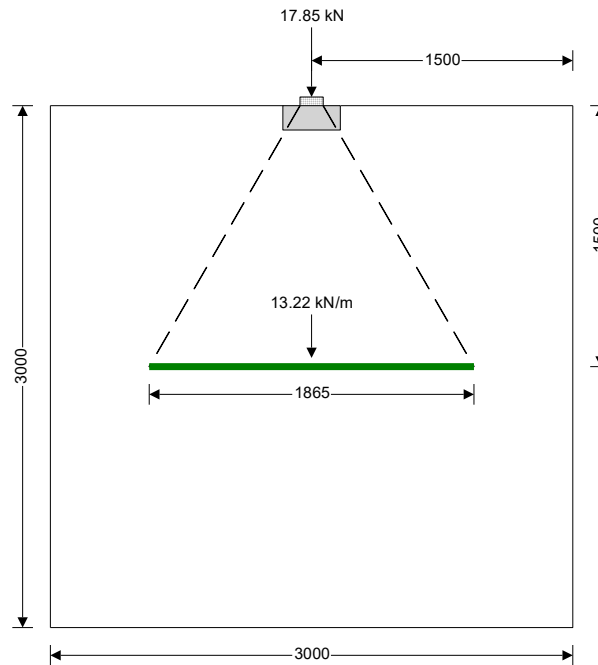
Slenderness ratio

$\lambda = h_{ef} / t_{ef} = 23.8$

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PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load	$G_{kc1} = 7.00 \text{ kN}$
Variable concentrated load	$Q_{kc1} = 5.60 \text{ kN}$
Eccentricity of concentrated load	$e_{c1} = 0 \text{ mm}$
Length of concentrated load	$L_{c1} = 133 \text{ mm}$
Width of concentrated load	$w_{c1} = 100 \text{ mm}$
Height of concentrated load	$h_{c1} = 3000 \text{ mm}$
Distance of load to right vertical edge	$r_{11} = 1434 \text{ mm}$
Distance of load to nearest vertical edge	$a_{11} = 1434 \text{ mm}$

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	$A_{b1} = L_{c1} \times w_{c1} = \mathbf{13300 \text{ mm}^2}$
Effective length of bearing at mid-height	$l_{efm1} = L_{c1} + h_{c1} \times \tan(30) = \mathbf{1865 \text{ mm}}$
Effective bearing area	$A_{ef1} = l_{efm1} \times t = \mathbf{186505 \text{ mm}^2}$
Bearing area ratio check	$A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = \mathbf{0.07}$
Initial enhancement factor	$\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = \mathbf{1.63}$
Maximum enhancement factor	$\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = \mathbf{1.49}$
Enhancement factor for concentrated loads	$\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1}) = \mathbf{1.49}$
Design value of the concentrated load	$N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ} = \mathbf{17.85 \text{ kN}}$
Design value concentrated load resistance	$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d = \mathbf{37.73 \text{ kN}}$

PASS - Design resistance exceeds applied concentrated load

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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

Krilov's functions at the point load

Using method of initial conditions

Initial moment of LH edge

Initial shear of LH edge

Which gives

and

Therefore,

Initial deflection of LH edge

Initial rotation of LH edge

Location of maximum deflection

Krilov's functions at the spreader length

Distance of point load right of loaction

Krilov's functions at the spreader length

Particular integral due to load

Maximum deflection

Concrete padstone

Point load

$$P_{11} = 165 \text{ mm}$$

$$L_{sp1} = 330 \text{ mm}$$

$$h_{sp1} = 140 \text{ mm}$$

$$w_{sp1} = 100 \text{ mm}$$

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

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

$$I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3 = 22866667 \text{ mm}^4$$

$$k_0 = E_w / h = 1.91 \text{ N/mm}^2/\text{mm}$$

$$K_{c1} = k_0 \times w_{sp1} = 190.51 \text{ N/mm/mm}$$

$$\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4} = 0.00289 \text{ mm}^{-1}$$

$$\alpha L_1 = \alpha_1 \times L_{sp1} = 0.95 \text{ Medium}$$

$$B_{al1} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = 0.93$$

$$C_{al1} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) = 0.45$$

$$D_{al1} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = 0.14$$

$$A_{\alpha P11} = \cosh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi) = 0.99$$

$$B_{\alpha P11} = 1/2 \times (\cosh(\alpha_1 \times P_{11}) \times \sin(180 \times \alpha_1 \times P_{11} / \pi) + \sinh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi)) = 0.48$$

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

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

$$(4 \times \alpha_1^2 \times C_{al1} \times \delta_{01} + 4 \times \alpha_1 \times D_{al1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - B_{\alpha P11} / \alpha_1 \times N_{Edc1} = 0.00 \text{ kNm}$$

$$(4 \times \alpha_1^3 \times B_{al1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{al1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - A_{\alpha P11} \times N_{Edc1} = 0.00 \text{ kN}$$

$$\delta_{01} = 0.27957 \text{ mm}$$

$$\Phi_{01} = 0.000059$$

$$x_{def1} = 165 \text{ mm}$$

$$A_{\alpha xdef1} = \cosh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi) = 0.99$$

$$B_{\alpha xdef1} = 1/2 \times (\cosh(\alpha_1 \times x_{def1}) \times \sin(180 \times \alpha_1 \times x_{def1} / \pi) + \sinh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi)) = 0.48$$

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

$$D_{ap1def1} = 1/4 \times (\cosh(\alpha_1 \times p_{1def1}) \times \sin(180 \times \alpha_1 \times p_{1def1} / \pi) - \sinh(\alpha_1 \times p_{1def1}) \times \cos(180 \times \alpha_1 \times p_{1def1} / \pi)) = 0.00$$

$$\delta'_{11} = D_{ap1def1} / \alpha_1^3 \times N_{Edc1} / (I_{sp1} \times E_{sp1}) = 0.000 \text{ mm}$$

$$\delta_{max1} = A_{\alpha xdef1} \times \delta_{01} + B_{\alpha xdef1} \times \Phi_{01} / \alpha_1 + \delta'_{11} = 0.287 \text{ mm}$$

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Location of maximum moment

$$x_{M1} = 165 \text{ mm}$$

Krilov's functions at the spreader length

$$C_{\alpha x M1} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) = 0.11$$

$$D_{\alpha x M1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi)) = 0.02$$

Distance of point load right of loaction

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

Krilov's functions at the spreader length

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

Particular integral due to load

$$M'_1 = -B_{\alpha p 1M1} / \alpha_1 \times N_{Edc1} = 0.00 \text{ kNm}$$

Maximum moment

$$M_{Edsp1} = (4 \times \alpha_1^2 \times C_{\alpha x M1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha x M1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M'_1 = 0.73 \text{ kNm}$$

Location of maximum shear

$$x_{V1} = 165 \text{ mm}$$

Krilov's functions at the spreader length

$$B_{\alpha x V1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi)) = 0.48$$

$$C_{\alpha x V1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) = 0.11$$

Distance of point load right of loaction

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

Krilov's functions at the spreader length

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

Particular integral due to load

$$V'_1 = -A_{\alpha p 1V1} \times N_{Edc1} = -17.85 \text{ kN}$$

Shear at concentrated point load

$$V_1 = (4 \times \alpha_1^3 \times B_{\alpha x V1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha x V1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + V'_1 = -8.93 \text{ kN}$$

Maximum shear

$$V_{Edsp1} = \text{Max}(\text{Abs}(V_1), N_{Edc1} - \text{Abs}(V_1)) = 8.93 \text{ kN}$$

Maximum allowable stress under spreader

$$\sigma_{Rdsp1} = \beta_1 \times f_d = 2.84 \text{ N/mm}^2$$

Maximum reaction

$$N_{Edsp1} = K_{c1} \times \delta_{\max 1} = 54.64 \text{ kN/m}$$

Design stress

$$\sigma_{Edsp1} = N_{Edsp1} / W_{sp1} = 0.55 \text{ N/mm}^2$$

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

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

Eccentricity of variable UDL at mid-height below concentrated load

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

Eccentricity of concentrated load at mid-height

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

Initial eccentricity - cl.5.5.1.1(4)

$$e_{init} = h_{ef} / 450 = 6.7 \text{ mm}$$

Concentrated load at mid-height as UDL

$$N_{mc1} = N_{Edc1} / l_{efm1} = 9.57 \text{ kN/m}$$

Vertical load at mid-height

$$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = 13.22 \text{ kN/m}$$

Design moment at mid-height

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

Eccentricities due to loads - eq. 6.7

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = 6.7 \text{ mm}$$

Slenderness ratio limit for creep eccentricity

$$\lambda_c = 27$$

Eccentricity due to creep

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

Eccentricity at mid-height - eq. 6.6

$$e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \times t) = 6.7 \text{ mm}$$

From eq. G2

$$A_{11} = 1 - 2 \times e_{mk1} / t = 0.87$$

From eq. G3

$$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = 1.06$$

Capacity reduction factor - eq. G1

$$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) = 0.50$$

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Design vertical resistance of panel - eq.6.2

$N_{Rd1} = \Phi_{m1} \times t \times f_d = \mathbf{94.32 \text{ kN/m}}$

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	46.8 kN	41.7 kN	1.44 N/mm ²	2.86 N/mm ²	0.502	Pass

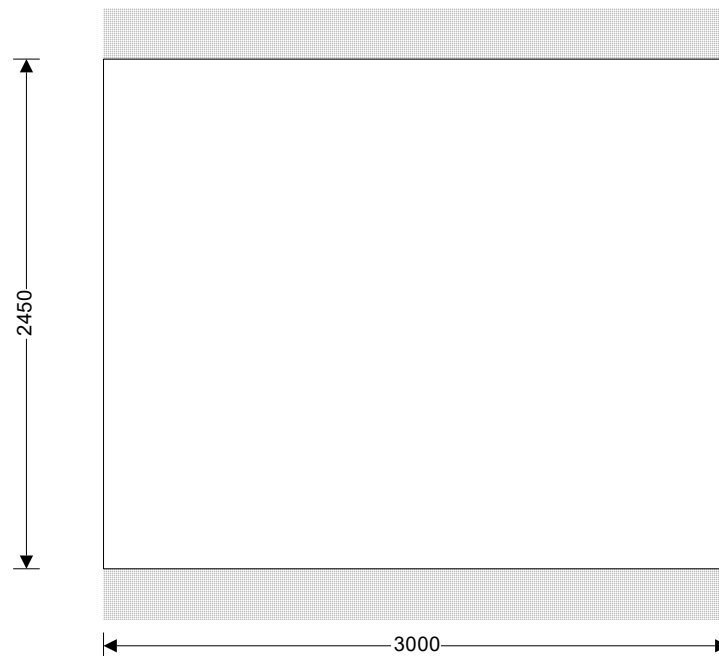
Masonry panel details

Panel length L = 3000 mm

Panel height h = 2450 mm

Panel support conditions

Top and bottom edges supported only



Effective height of masonry wall - Section 5.5.1.2

Vertical restraints

Wall restrained at top and bottom only

Horizontal restraints

Simply supported

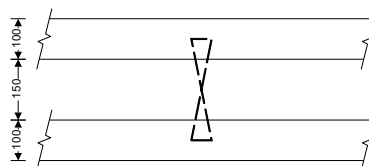
Reduction factor - eq. 5.5

$\rho_2 = 1.00$

Effective height of wall - eq. 5.2

$h_{ef} = \rho_2 \times h = 2450 \text{ mm}$

Wall construction details



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Wall type

Cavity wall

Inner leaf thickness

$t_2 = 100.0$ mm

Cavity thickness

$t_c = 150.0$ 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

$k_{tef} = 1.00$

Effective thickness - eq. 5.11

$t_{ef} = (k_{tef} \times t_{ef1}^3 + t_{ef2}^3)^{1/3} = 126.0$ mm

Masonry material details

Unit type

Aggregate concrete - Group 1

Compressive strength of masonry unit

$f_c = 7.3$ N/mm²

Height of unit

$h_u = 215$ mm

Width of unit

$w_u = 102$ mm

Conditioning factor

$k = 1.0$

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

Shape factor - Table A.1

$d_{sf} = 1.376$

Mean compressive strength of masonry unit

$f_b = f_c \times k \times d_{sf} = 10.04$ N/mm²

Specific weight of units

$\gamma = 18$ kN/m³

Mortar type

M4 - General Purpose

Compressive strength of mortar

$f_m = 4.0$ N/mm²

Compressive strength factor - Tbl. NA 4

$K = 0.75$

Characteristic compressive strength - eq. 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3} = 5.72$ N/mm²

Short term secant modulus of elasticity factor

$K_E = 1000$

Modulus of elasticity - cl.3.7.2

$E_w = K_E \times f_k = 5715$ N/mm²

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

$A = L \times t = 0.30$ m²

Design compressive strength of masonry

$f_d = f_k / \gamma_M = 1.91$ N/mm²

Partial safety factors for design loads

Partial safety factor for permanent load

$\gamma_{FG} = 1.35$

Partial safety factor for variable load

$\gamma_{FQ} = 1.50$

Superimposed vertical loading details

Permanent UDL at top of wall

$g_k = 0.00$ kN/m

Variable UDL at top of wall

$q_k = 0.00$ kN/m

Eccentricity of permanent UDL load

$e_{gu} = 0$ mm

Eccentricity of variable UDL load

$e_{qu} = 0$ mm

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit

$\lambda_{lim} = 27$

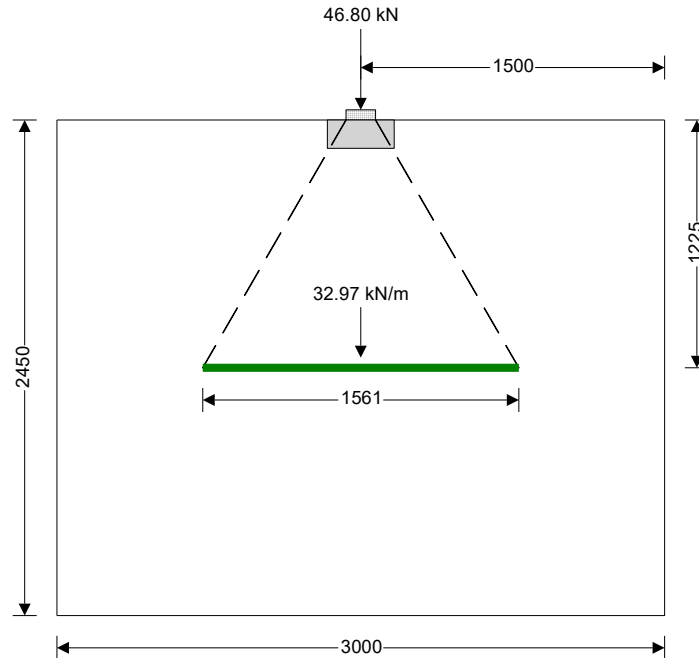
Slenderness ratio

$\lambda = h_{ef} / t_{ef} = 19.4$

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PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load	$G_{kc1} = 13.00 \text{ kN}$
Variable concentrated load	$Q_{kc1} = 19.50 \text{ kN}$
Eccentricity of concentrated load	$e_{c1} = 0 \text{ mm}$
Length of concentrated load	$L_{c1} = 146 \text{ mm}$
Width of concentrated load	$w_{c1} = 100 \text{ mm}$
Height of concentrated load	$h_{c1} = 2450 \text{ mm}$
Distance of load to right vertical edge	$r_{11} = 1427 \text{ mm}$
Distance of load to nearest vertical edge	$a_{11} = 1427 \text{ mm}$

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	$A_{b1} = L_{c1} \times w_{c1} = 14600 \text{ mm}^2$
Effective length of bearing at mid-height	$l_{efm1} = L_{c1} + h_{c1} \times \tan(30) = 1561 \text{ mm}$
Effective bearing area	$A_{ef1} = l_{efm1} \times t = 156051 \text{ mm}^2$
Bearing area ratio check	$A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = 0.09$
Initial enhancement factor	$\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = 1.64$
Maximum enhancement factor	$\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = 1.50$
Enhancement factor for concentrated loads	$\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1}) = 1.50$
Design value of the concentrated load	$N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ} = 46.80 \text{ kN}$
Design value concentrated load resistance	$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d = 41.72 \text{ kN}$

Applied concentrated load exceeds design resistance, spreader required!

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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

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Krilov's functions at the point load

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Initial moment of LH edge

Initial shear of LH edge

Which gives

and

Therefore,

Initial deflection of LH edge

Initial rotation of LH edge

Location of maximum deflection

Krilov's functions at the spreader length

Distance of point load right of loaction

Krilov's functions at the spreader length

Particular integral due to load

Maximum deflection

Concrete padstone

Point load

$$P_{11} = 165 \text{ mm}$$

$$L_{sp1} = 330 \text{ mm}$$

$$h_{sp1} = 140 \text{ mm}$$

$$w_{sp1} = 100 \text{ mm}$$

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

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

$$I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3 = 22866667 \text{ mm}^4$$

$$k_0 = E_w / h = 2.33 \text{ N/mm}^2/\text{mm}$$

$$K_{c1} = k_0 \times w_{sp1} = 233.28 \text{ N/mm/mm}$$

$$\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4} = 0.00304 \text{ mm}^{-1}$$

$$\alpha L_1 = \alpha_1 \times L_{sp1} = 1.00 \text{ Medium}$$

$$B_{al1} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = 0.97$$

$$C_{al1} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) = 0.50$$

$$D_{al1} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = 0.17$$

$$A_{\alpha P11} = \cosh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi) = 0.99$$

$$B_{\alpha P11} = 1/2 \times (\cosh(\alpha_1 \times P_{11}) \times \sin(180 \times \alpha_1 \times P_{11} / \pi) + \sinh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi)) = 0.50$$

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

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

$$(4 \times \alpha_1^2 \times C_{al1} \times \delta_{01} + 4 \times \alpha_1 \times D_{al1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - B_{\alpha P11} / \alpha_1 \times N_{Edc1} = 0.00 \text{ kNm}$$

$$(4 \times \alpha_1^3 \times B_{al1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{al1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - A_{\alpha P11} \times N_{Edc1} = 0.00 \text{ kN}$$

$$\delta_{01} = 0.59653 \text{ mm}$$

$$\Phi_{01} = 0.000154$$

$$x_{def1} = 165 \text{ mm}$$

$$A_{\alpha xdef1} = \cosh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi) = 0.99$$

$$B_{\alpha xdef1} = 1/2 \times (\cosh(\alpha_1 \times x_{def1}) \times \sin(180 \times \alpha_1 \times x_{def1} / \pi) + \sinh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi)) = 0.50$$

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

$$D_{\alpha p1def1} = 1/4 \times (\cosh(\alpha_1 \times p_{1def1}) \times \sin(180 \times \alpha_1 \times p_{1def1} / \pi) - \sinh(\alpha_1 \times p_{1def1}) \times \cos(180 \times \alpha_1 \times p_{1def1} / \pi)) = 0.00$$

$$\delta'_{11} = D_{\alpha p1def1} / \alpha_1^3 \times N_{Edc1} / (I_{sp1} \times E_{sp1}) = 0.000 \text{ mm}$$

$$\delta_{max1} = A_{\alpha xdef1} \times \delta_{01} + B_{\alpha xdef1} \times \Phi_{01} / \alpha_1 + \delta'_{11} = 0.616 \text{ mm}$$

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Location of maximum moment

Krilov's functions at the spreader length

$$x_{M1} = 165 \text{ mm}$$

$$C_{\alpha x M1} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) = 0.13$$

$$D_{\alpha x M1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi)) = 0.02$$

Distance of point load right of loaction

Krilov's functions at the spreader length

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

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

Particular integral due to load

$$M'_1 = -B_{\alpha p 1M1} / \alpha_1 \times N_{Edc1} = 0.00 \text{ kNm}$$

Maximum moment

$$M_{Edsp1} = (4 \times \alpha_1^2 \times C_{\alpha x M1} \times \delta_{01} + 4 \times \alpha_1 \times D_{\alpha x M1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M'_1 = 1.92 \text{ kNm}$$

Location of maximum shear

Krilov's functions at the spreader length

$$x_{V1} = 165 \text{ mm}$$

$$B_{\alpha x V1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi)) = 0.50$$

$$C_{\alpha x V1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) = 0.13$$

Distance of point load right of loaction

Krilov's functions at the spreader length

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

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

Particular integral due to load

$$V'_1 = -A_{\alpha p 1V1} \times N_{Edc1} = -46.80 \text{ kN}$$

Shear at concentrated point load

$$V_1 = (4 \times \alpha_1^3 \times B_{\alpha x V1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{\alpha x V1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + V'_1 = -23.40 \text{ kN}$$

Maximum shear

$$V_{Edsp1} = \text{Max}(\text{Abs}(V_1), N_{Edc1} - \text{Abs}(V_1)) = 23.40 \text{ kN}$$

Maximum allowable stress under spreader

$$\sigma_{Rdsp1} = \beta_1 \times f_d = 2.86 \text{ N/mm}^2$$

Maximum reaction

$$N_{Edsp1} = K_{c1} \times \delta_{max1} = 143.59 \text{ kN/m}$$

Design stress

$$\sigma_{Edsp1} = N_{Edsp1} / W_{sp1} = 1.44 \text{ N/mm}^2$$

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

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

Eccentricity of variable UDL at mid-height below concentrated load

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

Eccentricity of concentrated load at mid-height

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

Initial eccentricity - cl.5.5.1.1(4)

$$e_{init} = h_{ef} / 450 = 5.4 \text{ mm}$$

Concentrated load at mid-height as UDL

$$N_{mc1} = N_{Edc1} / l_{efm1} = 29.99 \text{ kN/m}$$

Vertical load at mid-height

$$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = 32.97 \text{ kN/m}$$

Design moment at mid-height

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

Eccentricities due to loads - eq. 6.7

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = 5.4 \text{ mm}$$

Slenderness ratio limit for creep eccentricity

$$\lambda_c = 27$$

Eccentricity due to creep

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

Eccentricity at mid-height - eq. 6.6

$$e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \times t) = 5.4 \text{ mm}$$

From eq. G2

$$A_{11} = 1 - 2 \times e_{mk1} / t = 0.89$$

From eq. G3

$$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = 0.83$$

Capacity reduction factor - eq. G1

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

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Design vertical resistance of panel - eq.6.2

$N_{Rd1} = \Phi_{m1} \times t \times f_d = \mathbf{120.46\text{ kN/m}}$

PASS - Design value of vertical resistance exceeds applied vertical load

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MASONRY BEARING DESIGN

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

Tedds calculation version 1.0.14

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	39.1 kN	41.7 kN	1.20 N/mm ²	2.86 N/mm ²	0.419	Pass

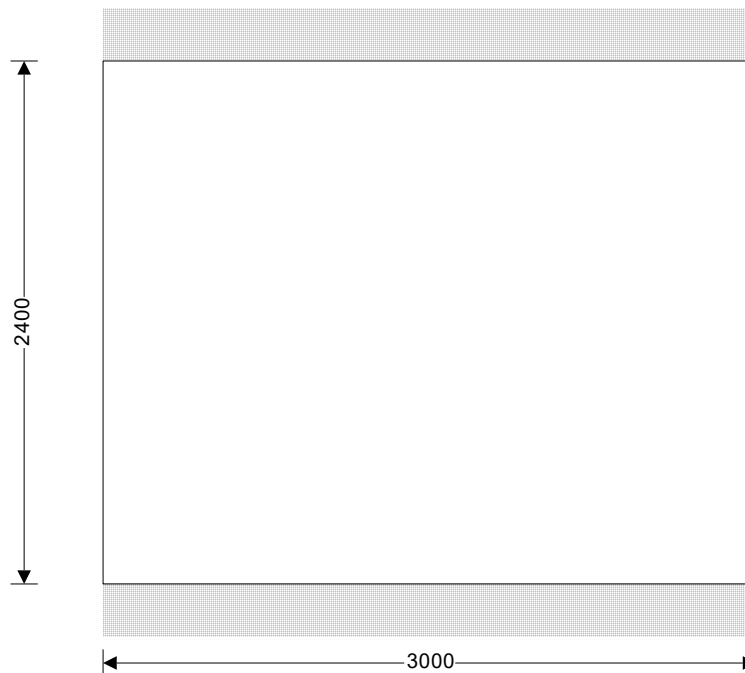
Masonry panel details

Panel length L = 3000 mm

Panel height h = 2400 mm

Panel support conditions

Top and bottom edges supported only



Effective height of masonry wall - Section 5.5.1.2

Vertical restraints

Wall restrained at top and bottom only

Horizontal restraints

Simply supported

Reduction factor - eq. 5.5

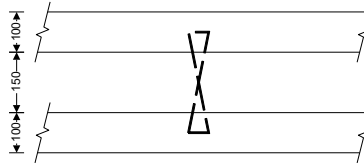
$\rho_2 = 1.00$

Effective height of wall - eq. 5.2

$h_{ef} = \rho_2 \times h = 2400 \text{ mm}$

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Wall construction details



Wall type

Cavity wall

Inner leaf thickness

$t_2 = 100.0$ mm

Cavity thickness

$t_c = 150.0$ 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

$k_{tef} = 1.00$

Effective thickness - eq. 5.11

$t_{ef} = (k_{tef} \times t_{ef1}^3 + t_{ef2}^3)^{1/3} = 126.0$ mm

Masonry material details

Aggregate concrete - Group 1

Unit type

$f_c = 7.3$ N/mm²

Compressive strength of masonry unit

Height of unit

$h_u = 215$ mm

Width of unit

$w_u = 102$ mm

Conditioning factor

$k = 1.0$

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

Shape factor - Table A.1

$d_{sf} = 1.376$

Mean compressive strength of masonry unit

$f_b = f_c \times k \times d_{sf} = 10.04$ N/mm²

Specific weight of units

$\gamma = 18$ kN/m³

Mortar type

M4 - General Purpose

Compressive strength of mortar

$f_m = 4.0$ N/mm²

Compressive strength factor - Tbl. NA 4

$K = 0.75$

Characteristic compressive strength - eq. 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3} = 5.72$ N/mm²

Short term secant modulus of elasticity factor

$K_E = 1000$

Modulus of elasticity - cl.3.7.2

$E_w = K_E \times f_k = 5715$ N/mm²

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

$A = L \times t = 0.30$ m²

Design compressive strength of masonry

$f_d = f_k / \gamma_M = 1.91$ N/mm²

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

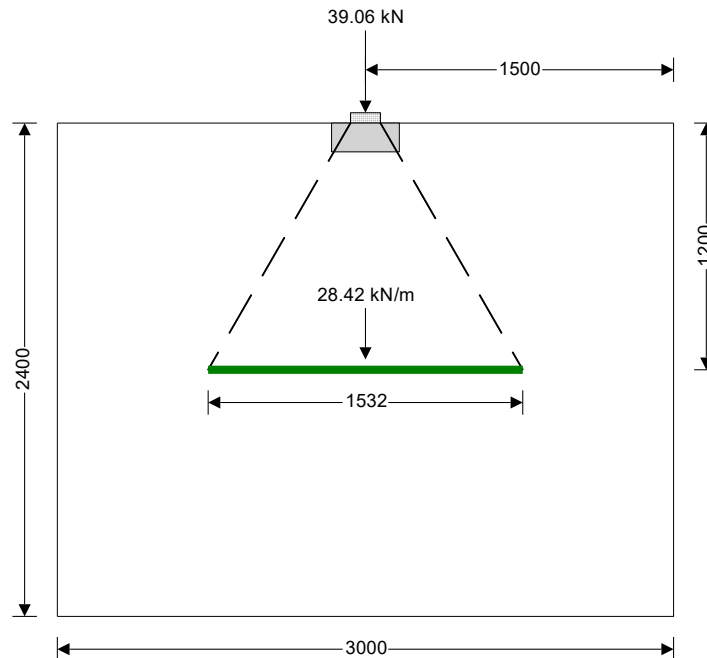
Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit $\lambda_{lim} = 27$

Slenderness ratio $\lambda = h_{ef} / t_{ef} = 19.0$

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details



Permanent concentrated load $G_{kc1} = 6.60 \text{ kN}$

Variable concentrated load $Q_{kc1} = 20.10 \text{ kN}$

Eccentricity of concentrated load $e_{c1} = 0 \text{ mm}$

Length of concentrated load $L_{c1} = 146 \text{ mm}$

Width of concentrated load $w_{c1} = 100 \text{ mm}$

Height of concentrated load $h_{c1} = 2400 \text{ mm}$

Distance of load to right vertical edge $r_{11} = 1427 \text{ mm}$

Distance of load to nearest vertical edge $a_{11} = 1427 \text{ mm}$

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 $A_{b1} = L_{c1} \times w_{c1} = 14600 \text{ mm}^2$

Effective length of bearing at mid-height $l_{efm1} = L_{c1} + h_{c1} \times \tan(30) = 1532 \text{ mm}$

Effective bearing area $A_{ef1} = l_{efm1} \times t = 153164 \text{ mm}^2$

Bearing area ratio check $A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = 0.10$

Initial enhancement factor $\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = 1.64$

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Maximum enhancement factor
Enhancement factor for concentrated loads
Design value of the concentrated load
Design value concentrated load resistance

$$\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = \mathbf{1.50}$$

$$\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1}) = \mathbf{1.50}$$

$$N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ} = \mathbf{39.06 \text{ kN}}$$

$$N_{Rdc1} = \beta_1 \times A_{b1} \times f_d = \mathbf{41.72 \text{ kN}}$$

PASS - Design resistance exceeds applied concentrated load

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

$$P_{11} = \mathbf{165 \text{ mm}}$$

$$L_{sp1} = \mathbf{330 \text{ mm}}$$

$$h_{sp1} = \mathbf{140 \text{ mm}}$$

$$w_{sp1} = \mathbf{100 \text{ mm}}$$

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

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

$$I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3 = \mathbf{22866667 \text{ mm}^4}$$

$$k_0 = E_w / h = \mathbf{2.38 \text{ N/mm}^2/\text{mm}}$$

$$K_{c1} = k_0 \times w_{sp1} = \mathbf{238.14 \text{ N/mm/mm}}$$

$$\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4} = \mathbf{0.00305 \text{ mm}^{-1}}$$

$$\alpha L_1 = \alpha_1 \times L_{sp1} = \mathbf{1.01 \text{ Medium}}$$

$$B_{al1} = 1/2 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) + \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = \mathbf{0.97}$$

$$C_{al1} = 1/2 \times \sinh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) = \mathbf{0.50}$$

$$D_{al1} = 1/4 \times (\cosh(\alpha L_1) \times \sin(180 \times \alpha L_1 / \pi) - \sinh(\alpha L_1) \times \cos(180 \times \alpha L_1 / \pi)) = \mathbf{0.17}$$

$$A_{\alpha P11} = \cosh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi) = \mathbf{0.99}$$

$$B_{\alpha P11} = 1/2 \times (\cosh(\alpha_1 \times P_{11}) \times \sin(180 \times \alpha_1 \times P_{11} / \pi) + \sinh(\alpha_1 \times P_{11}) \times \cos(180 \times \alpha_1 \times P_{11} / \pi)) = \mathbf{0.50}$$

Krilov's functions at the point load

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

$$(4 \times \alpha_1^2 \times C_{al1} \times \delta_{01} + 4 \times \alpha_1 \times D_{al1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - B_{\alpha P11} / \alpha_1 \times N_{Edc1} = \mathbf{0.00 \text{ kNm}}$$

and

$$(4 \times \alpha_1^3 \times B_{al1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{al1} \times \Phi_{01}) \times E_{sp1} \times I_{sp1} - A_{\alpha P11} \times N_{Edc1} = \mathbf{0.00 \text{ kN}}$$

Therefore,

Initial deflection of LH edge

$$\delta_{01} = \mathbf{0.48752 \text{ mm}}$$

Initial rotation of LH edge

$$\Phi_{01} = \mathbf{0.000128}$$

Location of maximum deflection

$$x_{def1} = \mathbf{165 \text{ mm}}$$

Krilov's functions at the spreader length

$$A_{\alpha x_{def1}} = \cosh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi) = \mathbf{0.99}$$

$$B_{\alpha x_{def1}} = 1/2 \times (\cosh(\alpha_1 \times x_{def1}) \times \sin(180 \times \alpha_1 \times x_{def1} / \pi) + \sinh(\alpha_1 \times x_{def1}) \times \cos(180 \times \alpha_1 \times x_{def1} / \pi)) = \mathbf{0.50}$$

Distance of point load right of loaction

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

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Krilov's functions at the spreader length

$$D_{ap1def1} = 1/4 \times (\cosh(\alpha_1 \times p_{1def1}) \times \sin(180 \times \alpha_1 \times p_{1def1} / \pi) - \sinh(\alpha_1 \times p_{1def1}) \times \cos(180 \times \alpha_1 \times p_{1def1} / \pi)) = \mathbf{0.00}$$

Particular integral due to load

$$\delta'_1 = D_{ap1def1} / \alpha_1^3 \times N_{Edc1} / (I_{sp1} \times E_{sp1}) = \mathbf{0.000 \text{ mm}}$$

Maximum deflection

$$\delta_{max1} = A_{axdef1} \times \delta_{01} + B_{axdef1} \times \Phi_{01} / \alpha_1 + \delta'_1 = \mathbf{0.503 \text{ mm}}$$

Location of maximum moment

$$x_{M1} = \mathbf{165 \text{ mm}}$$

Krilov's functions at the spreader length

$$C_{axM1} = 1/2 \times \sinh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) = \mathbf{0.13}$$

$$D_{axM1} = 1/4 \times (\cosh(\alpha_1 \times x_{M1}) \times \sin(180 \times \alpha_1 \times x_{M1} / \pi) - \sinh(\alpha_1 \times x_{M1}) \times \cos(180 \times \alpha_1 \times x_{M1} / \pi)) = \mathbf{0.02}$$

Distance of point load right of loaction

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

Krilov's functions at the spreader length

$$B_{ap1M1} = 1/2 \times (\cosh(\alpha_1 \times p_{1M1}) \times \sin(180 \times \alpha_1 \times p_{1M1} / \pi) + \sinh(\alpha_1 \times p_{1M1}) \times \cos(180 \times \alpha_1 \times p_{1M1} / \pi)) = \mathbf{0.00}$$

Particular integral due to load

$$M'_1 = -B_{ap1M1} / \alpha_1 \times N_{Edc1} = \mathbf{0.00 \text{ kNm}}$$

Maximum moment

$$M_{Edsp1} = (4 \times \alpha_1^2 \times C_{axM1} \times \delta_{01} + 4 \times \alpha_1 \times D_{axM1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + M'_1 = \mathbf{1.60 \text{ kNm}}$$

Location of maximum shear

$$x_{V1} = \mathbf{165 \text{ mm}}$$

Krilov's functions at the spreader length

$$B_{axV1} = 1/2 \times (\cosh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) + \sinh(\alpha_1 \times x_{V1}) \times \cos(180 \times \alpha_1 \times x_{V1} / \pi)) = \mathbf{0.50}$$

$$C_{axV1} = 1/2 \times \sinh(\alpha_1 \times x_{V1}) \times \sin(180 \times \alpha_1 \times x_{V1} / \pi) = \mathbf{0.13}$$

Distance of point load right of loaction

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

Krilov's functions at the spreader length

$$A_{ap1V1} = \cosh(\alpha_1 \times p_{1V1}) \times \cos(180 \times \alpha_1 \times p_{1V1} / \pi) = \mathbf{1.00}$$

Particular integral due to load

$$V'_1 = -A_{ap1V1} \times N_{Edc1} = \mathbf{-39.06 \text{ kN}}$$

Shear at concentrated point load

$$V_1 = (4 \times \alpha_1^3 \times B_{axV1} \times \delta_{01} + 4 \times \alpha_1^2 \times C_{axV1} \times \Phi_{01}) \times (I_{sp1} \times E_{sp1}) + V'_1 = \mathbf{-19.53 \text{ kN}}$$

Maximum shear

$$V_{Edsp1} = \text{Max}(\text{Abs}(V_1), N_{Edc1} - \text{Abs}(V_1)) = \mathbf{19.53 \text{ kN}}$$

Maximum allowable stress under spreader

$$\sigma_{Rdsp1} = \beta_1 \times f_d = \mathbf{2.86 \text{ N/mm}^2}$$

Maximum reaction

$$N_{Edsp1} = K_{c1} \times \delta_{max1} = \mathbf{119.88 \text{ kN/m}}$$

Design stress

$$\sigma_{Edsp1} = N_{Edsp1} / W_{sp1} = \mathbf{1.20 \text{ N/mm}^2}$$

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)

$$e_{init} = h_{ef} / 450 = \mathbf{5.3 \text{ mm}}$$

Concentrated load at mid-height as UDL

$$N_{mc1} = N_{Edc1} / l_{efm1} = \mathbf{25.50 \text{ kN/m}}$$

Vertical load at mid-height

$$N_{Ed1} = (g_k + \gamma \times t \times (h - h_{c1} / 2)) \times \gamma_{fG} + q_k \times \gamma_{fQ} + N_{mc1} = \mathbf{28.42 \text{ kN/m}}$$

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

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = \mathbf{5.3 \text{ mm}}$$

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

$$e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \times t) = \mathbf{5.3 \text{ mm}}$$

From eq. G2

$$A_{11} = 1 - 2 \times e_{mk1} / t = \mathbf{0.89}$$

From eq. G3

$$u_1 = (h_{ef} / t_{ef} \times (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \times e_{mk1} / t) = \mathbf{0.81}$$

Capacity reduction factor - eq. G1

$$\Phi_{m1} = A_{11} \times \exp(-(u_1^2) / 2) = \mathbf{0.64}$$

Design vertical resistance of panel - eq.6.2

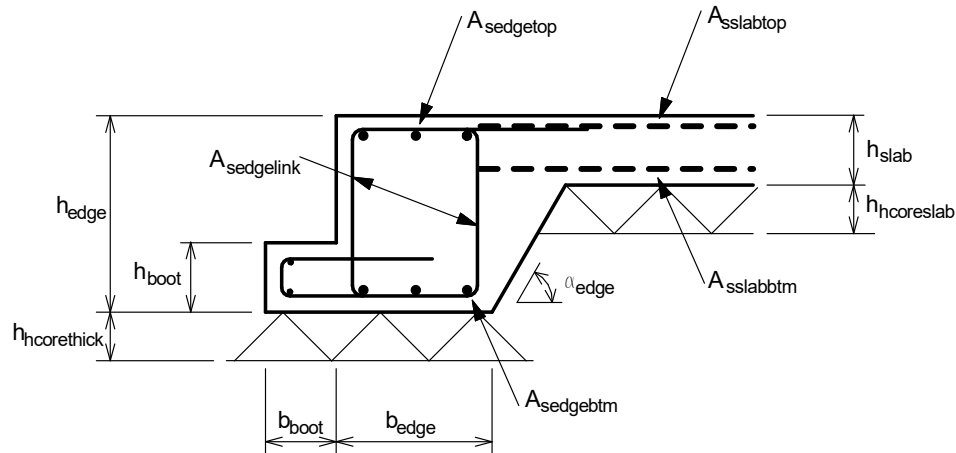
$$N_{Rd1} = \Phi_{m1} \times t \times f_d = \mathbf{122.80 \text{ kN/m}}$$

PASS - Design value of vertical resistance exceeds applied vertical load

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RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)

Tedds calculation version 1.0.13



Soil and raft definition

Soil definition

Allowable bearing pressure

$$q_{allow} = 50.0 \text{ kN/m}^2$$

Depth of hardcore beneath slab

$$h_{hcoreslab} = 150 \text{ mm} \text{ (Dispersal allowed for bearing pressure check)}$$

Depth of hardcore beneath thickenings

$$h_{hcorethick} = 150 \text{ mm} \text{ (Dispersal allowed for bearing pressure check)}$$

Density of hardcore

$$\gamma_{hcore} = 20.0 \text{ kN/m}^3$$

Basic assumed diameter of local depression

$$\phi_{depbasic} = 3000 \text{ mm}$$

Diameter under slab modified for hardcore

$$\phi_{depslab} = \phi_{depbasic} - h_{hcoreslab} = 2850 \text{ mm}$$

Diameter under thickenings modified for hardcore

$$\phi_{depthick} = \phi_{depbasic} - h_{hcorethick} = 2850 \text{ mm}$$

Raft slab definition

Max dimension/max dimension between joints

$$l_{max} = 20.000 \text{ m}$$

Slab thickness

$$h_{slab} = 225 \text{ mm}$$

Concrete strength

$$f_{cu} = 40 \text{ N/mm}^2$$

Poissons ratio of concrete

$$\nu = 0.2$$

Slab mesh reinforcement strength

$$f_{yslab} = 500 \text{ N/mm}^2$$

Partial safety factor for steel reinforcement

$$\gamma_s = 1.15$$

From C&CA document 'Concrete ground floors' Table 5

Minimum mesh required in top for shrinkage

$$A252$$

Actual mesh provided in top

$$2 \times A393 (A_{sslabbtop} = 786 \text{ mm}^2/\text{m})$$

Mesh provided in bottom

$$2 \times A393 (A_{sslabbtm} = 786 \text{ mm}^2/\text{m})$$

Top mesh bar diameter

$$\phi_{slabbtop} = 10 \text{ mm}$$

Bottom mesh bar diameter

$$\phi_{slabbtm} = 10 \text{ mm}$$

Cover to top reinforcement

$$C_{top} = 30 \text{ mm}$$

Cover to bottom reinforcement

$$C_{btm} = 50 \text{ mm}$$

Average effective depth of top reinforcement

$$d_{tslabav} = h_{slab} - C_{top} - \phi_{slabbtop} = 185 \text{ mm}$$

Average effective depth of bottom reinforcement

$$d_{bslabav} = h_{slab} - C_{btm} - \phi_{slabbtm} = 165 \text{ mm}$$

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Overall average effective depth

$$d_{slabav} = (d_{slabav} + d_{bslabav})/2 = \mathbf{175 \text{ mm}}$$

Minimum effective depth of top reinforcement

$$d_{slabmin} = d_{slabav} - \phi_{slabtop}/2 = \mathbf{180 \text{ mm}}$$

Minimum effective depth of bottom reinforcement

$$d_{bslabmin} = d_{bslabav} - \phi_{slabbtm}/2 = \mathbf{160 \text{ mm}}$$

Edge beam definition

Overall depth

$$h_{edge} = \mathbf{600 \text{ mm}}$$

Width

$$b_{edge} = \mathbf{350 \text{ mm}}$$

Depth of boot

$$h_{boot} = \mathbf{300 \text{ mm}}$$

Width of boot

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

Angle of chamfer to horizontal

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

Strength of main bar reinforcement

$$f_y = \mathbf{500 \text{ N/mm}^2}$$

Strength of link reinforcement

$$f_{ys} = \mathbf{500 \text{ N/mm}^2}$$

Reinforcement provided in top

$$\mathbf{5 \text{ H20 bars } (A_{sedgetop} = 1571 \text{ mm}^2)}$$

Reinforcement provided in bottom

$$\mathbf{5 \text{ H16 bars } (A_{sedgebtm} = 1005 \text{ mm}^2)}$$

Link reinforcement provided

$$\mathbf{2 \text{ H10 legs at 200 ctrs } (A_{sv/s_v} = 0.785 \text{ mm})}$$

Bottom cover to links

$$C_{beam} = \mathbf{50 \text{ mm}}$$

Effective depth of top reinforcement

$$d_{edgetop} = h_{edge} - C_{top} - \phi_{slabtop} - \phi_{edgelink} - \phi_{edgetop}/2 = \mathbf{540 \text{ mm}}$$

Effective depth of bottom reinforcement

$$d_{edgebtm} = h_{edge} - C_{beam} - \phi_{edgelink} - \phi_{edgebtm}/2 = \mathbf{532 \text{ mm}}$$

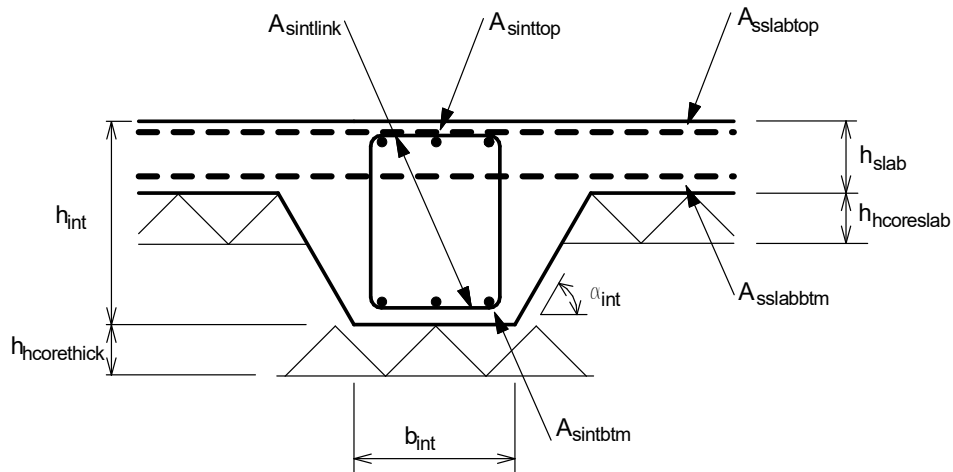
Boot main reinforcement

$$\mathbf{H10 \text{ bars at 200 ctrs } (A_{sboot} = 393 \text{ mm}^2/\text{m})}$$

Effective depth of boot reinforcement

$$d_{boot} = h_{boot} - C_{beam} - \phi_{boot}/2 = \mathbf{245 \text{ mm}}$$

Internal beam definition



Overall depth

$$h_{int} = \mathbf{600 \text{ mm}}$$

Width

$$b_{int} = \mathbf{500 \text{ mm}}$$

Angle of chamfer to horizontal

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

Strength of main bar reinforcement

$$f_y = \mathbf{500 \text{ N/mm}^2}$$

Strength of link reinforcement

$$f_{ys} = \mathbf{500 \text{ N/mm}^2}$$

Reinforcement provided in top

$$\mathbf{4 \text{ H16 bars } (A_{sinttop} = 804 \text{ mm}^2)}$$

Reinforcement provided in bottom

$$\mathbf{4 \text{ H16 bars } (A_{sintbtm} = 804 \text{ mm}^2)}$$

Link reinforcement provided

$$\mathbf{2 \text{ H10 legs at 200 ctrs } (A_{sv/s_v} = 0.785 \text{ mm})}$$

Effective depth of top reinforcement

$$d_{inttop} = h_{int} - C_{top} - 2 \times \phi_{slabtop} - \phi_{inttop}/2 = \mathbf{542 \text{ mm}}$$

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Effective depth of bottom reinforcement

$$d_{intbtm} = h_{int} - C_{beam} - \phi_{intlink} - \phi_{intbtm}/2 = 532 \text{ mm}$$

Internal slab design checks

Basic loading

Slab self weight

$$W_{slab} = 24 \text{ kN/m}^3 \times h_{slab} = 5.4 \text{ kN/m}^2$$

Hardcore

$$W_{hcoreslab} = \gamma_{hcore} \times h_{hcoreslab} = 3.0 \text{ kN/m}^2$$

Applied loading

Uniformly distributed dead load

$$W_{Dudl} = 2.5 \text{ kN/m}^2$$

Uniformly distributed live load

$$W_{Ludl} = 1.5 \text{ kN/m}^2$$

Internal slab bearing pressure check

Total uniform load at formation level

$$W_{udl} = W_{slab} + W_{hcoreslab} + W_{Dudl} + W_{Ludl} = 12.4 \text{ kN/m}^2$$

PASS - $w_{udl} \leq q_{allow}$ - Applied bearing pressure is less than allowable

Internal slab bending and shear check

Applied bending moments

Span of slab

$$l_{slab} = \phi_{depslab} + d_{tslabav} = 3035 \text{ mm}$$

Ultimate self weight udl

$$W_{swult} = 1.4 \times W_{slab} = 7.6 \text{ kN/m}^2$$

Self weight moment at centre

$$M_{csw} = W_{swult} \times l_{slab}^2 \times (1 + \nu) / 64 = 1.3 \text{ kNm/m}$$

Self weight moment at edge

$$M_{esw} = W_{swult} \times l_{slab}^2 / 32 = 2.2 \text{ kNm/m}$$

Self weight shear force at edge

$$V_{sw} = W_{swult} \times l_{slab} / 4 = 5.7 \text{ kN/m}$$

Moments due to applied uniformly distributed loads

Ultimate applied udl

$$W_{udlult} = 1.4 \times W_{Dudl} + 1.6 \times W_{Ludl} = 5.9 \text{ kN/m}^2$$

Moment at centre

$$M_{cudl} = W_{udlult} \times l_{slab}^2 \times (1 + \nu) / 64 = 1.0 \text{ kNm/m}$$

Moment at edge

$$M_{eudl} = W_{udlult} \times l_{slab}^2 / 32 = 1.7 \text{ kNm/m}$$

Shear force at edge

$$V_{udl} = W_{udlult} \times l_{slab} / 4 = 4.5 \text{ kN/m}$$

Resultant moments and shears

Total moment at edge

$$M_{\Sigma e} = 3.9 \text{ kNm/m}$$

Total moment at centre

$$M_{\Sigma c} = 2.3 \text{ kNm/m}$$

Total shear force

$$V_{\Sigma} = 10.2 \text{ kN/m}$$

Reinforcement required in top

K factor

$$K_{slabtop} = M_{\Sigma e} / (f_{cu} \times d_{tslabav}^2) = 0.003$$

Lever arm

$$Z_{slabtop} = d_{tslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtop}/0.9)}) = 175.8 \text{ mm}$$

Area of steel required for bending

$$A_{sslabtopbend} = M_{\Sigma e} / ((1.0/\gamma_s) \times f_{yslab} \times Z_{slabtop}) = 51 \text{ mm}^2/\text{m}$$

Minimum area of steel required

$$A_{sslabmin} = 0.0013 \times h_{slab} = 293 \text{ mm}^2/\text{m}$$

Area of steel required

$$A_{sslabtopreq} = \max(A_{sslabtopbend}, A_{sslabmin}) = 293 \text{ mm}^2/\text{m}$$

PASS - $A_{sslabtopreq} \leq A_{sslabtop}$ - Area of reinforcement provided in top to span local depressions is adequate

Reinforcement required in bottom

K factor

$$K_{slabbtm} = M_{\Sigma c} / (f_{cu} \times d_{bslabav}^2) = 0.002$$

Lever arm

$$Z_{slabbtm} = d_{bslabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbtm}/0.9)}) = 156.7 \text{ mm}$$

Area of steel required for bending

$$A_{sslabbtmbend} = M_{\Sigma c} / ((1.0/\gamma_s) \times f_{yslab} \times Z_{slabbtm}) = 34 \text{ mm}^2/\text{m}$$

Area of steel required

$$A_{sslabbtmreq} = \max(A_{sslabbtmbend}, A_{sslabmin}) = 293 \text{ mm}^2/\text{m}$$

PASS - $A_{sslabbtmreq} \leq A_{sslabbtm}$ - Area of reinforcement provided in bottom to span local depressions is adequate

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Shear check

Applied shear stress

$$v = V_z/d_{tslabmin} = \mathbf{0.057 \text{ N/mm}^2}$$

Tension steel ratio

$$\rho = 100 \times A_{sslabtop}/d_{tslabmin} = \mathbf{0.437}$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$v_c = \mathbf{0.685 \text{ N/mm}^2}$$

PASS - $v \leq v_c$ - Shear capacity of the slab is adequate

Internal slab deflection check

Basic allowable span to depth ratio

$$\text{Ratio}_{\text{basic}} = \mathbf{26.0}$$

Moment factor

$$M_{\text{factor}} = M_{\Sigma c}/d_{bslabav}^2 = \mathbf{0.085 \text{ N/mm}^2}$$

Steel service stress

$$f_s = 2/3 \times f_{yslab} \times A_{sslabbtmbend}/A_{sslabbtm} = \mathbf{14.466 \text{ N/mm}^2}$$

Modification factor

$$MF_{\text{slab}} = \min(2.0, 0.55 + [(477 \text{ N/mm}^2 - f_s)/(120 \times (0.9 \text{ N/mm}^2 + M_{\text{factor}})]))$$

$$MF_{\text{slab}} = \mathbf{2.000}$$

Modified allowable span to depth ratio

$$\text{Ratio}_{\text{allow}} = \text{Ratio}_{\text{basic}} \times MF_{\text{slab}} = \mathbf{52.000}$$

Actual span to depth ratio

$$\text{Ratio}_{\text{actual}} = l_{\text{slab}}/d_{bslabav} = \mathbf{18.394}$$

PASS - $\text{Ratio}_{\text{actual}} \leq \text{Ratio}_{\text{allow}}$ - Slab span to depth ratio is adequate

Edge beam design checks

Basic loading

Hardcore

$$W_{\text{corethick}} = \gamma_{\text{hcore}} \times h_{\text{corethick}} = \mathbf{3.0 \text{ kN/m}^2}$$

Edge beam

Rectangular beam element

$$W_{\text{beam}} = 24 \text{ kN/m}^3 \times h_{\text{edge}} \times b_{\text{edge}} = \mathbf{5.0 \text{ kN/m}}$$

Boot element

$$W_{\text{boot}} = 24 \text{ kN/m}^3 \times h_{\text{boot}} \times b_{\text{boot}} = \mathbf{1.4 \text{ kN/m}}$$

Chamfer element

$$W_{\text{chamfer}} = 24 \text{ kN/m}^3 \times (h_{\text{edge}} - h_{\text{slab}})^2/(2 \times \tan(\alpha_{\text{edge}})) = \mathbf{1.0 \text{ kN/m}}$$

Slab element

$$W_{\text{slabelm}} = 24 \text{ kN/m}^3 \times h_{\text{slab}} \times (h_{\text{edge}} - h_{\text{slab}})/\tan(\alpha_{\text{edge}}) = \mathbf{1.2 \text{ kN/m}}$$

Edge beam self weight

$$W_{\text{edge}} = W_{\text{beam}} + W_{\text{boot}} + W_{\text{chamfer}} + W_{\text{slabelm}} = \mathbf{8.6 \text{ kN/m}}$$

Edge load number 1

Load type

Longitudinal line load

$$W_{\text{Dedge1}} = \mathbf{34.7 \text{ kN/m}}$$

Dead load

$$W_{\text{Ledge1}} = \mathbf{10.2 \text{ kN/m}}$$

Live load

Ultimate load

$$W_{\text{ultedge1}} = 1.4 \times W_{\text{Dedge1}} + 1.6 \times W_{\text{Ledge1}} = \mathbf{64.8 \text{ kN/m}}$$

Longitudinal line load width

$$b_{\text{edge1}} = \mathbf{350 \text{ mm}}$$

Centroid of load from outside face of raft

$$x_{\text{edge1}} = \mathbf{175 \text{ mm}}$$

Edge beam bearing pressure check

Effective bearing width of edge beam

$$b_{\text{bearing}} = b_{\text{edge}} + b_{\text{boot}} + (h_{\text{edge}} - h_{\text{slab}})/\tan(\alpha_{\text{edge}}) = \mathbf{767 \text{ mm}}$$

Total uniform load at formation level

$$W_{\text{udedge}} = W_{\text{Dudl}} + W_{\text{Ludl}} + W_{\text{edge}}/b_{\text{bearing}} + W_{\text{hcorethick}} = \mathbf{18.3 \text{ kN/m}^2}$$

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

Load x distance for edge load 1

$$\text{Moment}_1 = W_{\text{ultedge1}} \times x_{\text{edge1}} = \mathbf{11.3 \text{ kN}}$$

Sum of ultimate longitud'l and equivalent line loads $\Sigma \text{UDL} = \mathbf{64.8 \text{ kN/m}}$

Sum of load x distances

$$\Sigma \text{Moment} = \mathbf{11.3 \text{ kN}}$$

Centroid of loads

$$x_{\text{bar}} = \Sigma \text{Moment}/\Sigma \text{UDL} = \mathbf{175 \text{ mm}}$$

Initially assume no moment transferred into slab due to load/reaction eccentricity

Sum of unfactored longitud'l and eff'tive line loads $\Sigma \text{UDLsls} = \mathbf{44.9 \text{ kN/m}}$

Allowable bearing width

$$b_{\text{allow}} = 2 \times x_{\text{bar}} + 2 \times h_{\text{hcoreslab}} \times \tan(30) = \mathbf{523 \text{ mm}}$$

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Bearing pressure due to line/point loads

$$q_{linepoint} = \Sigma UDLs / b_{allow} = \mathbf{85.8 \text{ kN/m}^2}$$

Total applied bearing pressure

$$q_{edge} = q_{linepoint} + W_{udledge} = \mathbf{104.0 \text{ kN/m}^2}$$

$q_{edge} > q_{allow}$ - The slab is required to resist a moment due to eccentricity

Now assume moment due to load/reaction eccentricity is resisted by slab

Bearing width required

$$b_{req} = \Sigma UDLs / (q_{allow} - W_{udledge}) = \mathbf{1413 \text{ mm}}$$

Effective bearing width at u/s of slab

$$b_{reqeff} = b_{req} - 2 \times h_{coreslab} \times \tan(30) = \mathbf{1240 \text{ mm}}$$

Load/reaction eccentricity

$$e = b_{reqeff} / 2 - x_{bar} = \mathbf{445 \text{ mm}}$$

Ultimate moment to be resisted by slab

$$M_{ecc} = \Sigma UDL \times e = \mathbf{28.9 \text{ kNm/m}}$$

From slab bending check

Moment due to depression under slab (hogging)

$$M_{\Sigma e} = \mathbf{3.9 \text{ kNm/m}}$$

Total moment to be resisted by slab top steel

$$M_{slabtop} = M_{ecc} + M_{\Sigma e} = \mathbf{32.7 \text{ kNm/m}}$$

K factor

$$K_{slab} = M_{slabtop} / (f_{cu} \times d_{slabmin}^2) = \mathbf{0.025}$$

Lever arm

$$Z_{slab} = d_{slabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slab}/0.9)}) = \mathbf{171 \text{ mm}}$$

Area of steel required

$$A_{sslabreq} = M_{slabtop} / ((1.0/\gamma_s) \times f_y \times Z_{slab}) = \mathbf{440 \text{ mm}^2/\text{m}}$$

PASS - $A_{sslabreq} \leq A_{sslabtop}$ - Area of reinforcement provided to transfer moment into slab is adequate

The allowable bearing pressure under the edge beam will not be exceeded

Library item - B pressure with moment transfer

Edge beam bending check

Divider for moments due to udl's

$$\beta_{udl} = \mathbf{10.0}$$

Applied bending moments

Span of edge beam

$$l_{edge} = \phi_{depthick} + d_{edgetop} = \mathbf{3390 \text{ mm}}$$

Ultimate self weight udl

$$W_{edgeult} = 1.4 \times W_{edge} = \mathbf{12.1 \text{ kN/m}}$$

Ultimate slab udl (approx)

$$W_{edgeslab} = \max(0 \text{ kN/m}, 1.4 \times W_{slab} \times ((\phi_{depthick}/2 \times 3/4) - (b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}))))$$

$$W_{edgeslab} = \mathbf{3.8 \text{ kN/m}}$$

Self weight and slab bending moment

$$M_{edgesw} = (W_{edgeult} + W_{edgeslab}) \times l_{edge}^2 / \beta_{udl} = \mathbf{18.2 \text{ kNm}}$$

Self weight shear force

$$V_{edgesw} = (W_{edgeult} + W_{edgeslab}) \times l_{edge} / 2 = \mathbf{26.9 \text{ kN}}$$

Moments due to applied uniformly distributed loads

Ultimate udl (approx)

$$W_{edgeudl} = W_{udlult} \times \phi_{depthick} / 2 \times 3/4 = \mathbf{6.3 \text{ kN/m}}$$

Bending moment

$$M_{edgeudl} = W_{edgeudl} \times l_{edge}^2 / \beta_{udl} = \mathbf{7.2 \text{ kNm}}$$

Shear force

$$V_{edgeudl} = W_{edgeudl} \times l_{edge} / 2 = \mathbf{10.7 \text{ kN}}$$

Moment and shear due to load number 1

Bending moment

$$M_{edge1} = W_{ultedge1} \times l_{edge}^2 / \beta_{udl} = \mathbf{74.5 \text{ kNm}}$$

Shear force

$$V_{edge1} = W_{ultedge1} \times l_{edge} / 2 = \mathbf{109.9 \text{ kN}}$$

Resultant moments and shears

Total moment (hogging and sagging)

$$M_{\Sigma edge} = \mathbf{100.0 \text{ kNm}}$$

Maximum shear force

$$V_{\Sigma edge} = \mathbf{147.5 \text{ kN}}$$

Reinforcement required in top

Width of section in compression zone

$$b_{edgetop} = b_{edge} + b_{boot} = \mathbf{550 \text{ mm}}$$

Average web width

$$b_w = b_{edge} + (h_{edge} / \tan(\alpha_{edge})) / 2 = \mathbf{523 \text{ mm}}$$

K factor

$$K_{edgetop} = M_{\Sigma edge} / (f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = \mathbf{0.016}$$

Lever arm

$$Z_{edgetop} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgetop}/0.9)}) = \mathbf{513 \text{ mm}}$$

Area of steel required for bending

$$A_{sedgetopbend} = M_{\Sigma edge} / ((1.0/\gamma_s) \times f_y \times Z_{edgetop}) = \mathbf{448 \text{ mm}^2}$$

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Minimum area of steel required

$$A_{sedge\ top\ min} = 0.0013 \times 1.0 \times b_w \times h_{edge} = \mathbf{408\ mm^2}$$

Area of steel required

$$A_{sedge\ top\ req} = \max(A_{sedge\ top\ bend}, A_{sedge\ top\ min}) = \mathbf{448\ mm^2}$$

PASS - $A_{sedge\ top\ req} \leq A_{sedge\ top}$ - Area of reinforcement provided in top of edge beams is adequate

Reinforcement required in bottom

Width of section in compression zone

$$b_{edge\ botm} = b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}) + 0.1 \times l_{edge} = \mathbf{906\ mm}$$

K factor

$$K_{edge\ botm} = M_{\Sigma edge}/(f_{cu} \times b_{edge\ botm} \times d_{edge\ botm}^2) = \mathbf{0.010}$$

Lever arm

$$Z_{edge\ botm} = d_{edge\ botm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edge\ botm}/0.9)}) = \mathbf{505\ mm}$$

Area of steel required for bending

$$A_{sedge\ botm\ bend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times Z_{edge\ botm}) = \mathbf{455\ mm^2}$$

Minimum area of steel required

$$A_{sedge\ botm\ min} = 0.0013 \times 1.0 \times b_w \times h_{edge} = \mathbf{408\ mm^2}$$

Area of steel required

$$A_{sedge\ botm\ req} = \max(A_{sedge\ botm\ bend}, A_{sedge\ botm\ min}) = \mathbf{455\ mm^2}$$

PASS - $A_{sedge\ botm\ req} \leq A_{sedge\ botm}$ - Area of reinforcement provided in bottom of edge beams is adequate

Edge beam shear check

Applied shear stress

$$V_{edge} = V_{\Sigma edge}/(b_w \times d_{edge\ top}) = \mathbf{0.522\ N/mm^2}$$

Tension steel ratio

$$\rho_{edge} = 100 \times A_{sedge\ top}/(b_w \times d_{edge\ top}) = \mathbf{0.556}$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$V_{cedge} = \mathbf{0.608\ N/mm^2}$$

$V_{edge} \leq V_{cedge} + 0.4\ N/mm^2$ - Therefore minimum links required

Link area to spacing ratio required

$$A_{sv_upon_svreqedge} = 0.4\ N/mm^2 \times b_w/((1.0/\gamma_s) \times f_{ys}) = \mathbf{0.481\ mm}$$

Link area to spacing ratio provided

$$A_{sv_upon_svproedge} = N_{edgelink} \times \pi \times \phi_{edgelink}^2/(4 \times S_{vedge}) = \mathbf{0.785\ mm}$$

PASS - $A_{sv_upon_svreqedge} \leq A_{sv_upon_svproedge}$ - Shear reinforcement provided in edge beams is adequate

Boot design check

Effective cantilever span

$$l_{boot} = b_{boot} + d_{boot}/2 = \mathbf{323\ mm}$$

Approximate ultimate bearing pressure

$$q_{ult} = 1.55 \times q_{allow} = \mathbf{77.5\ kN/m^2}$$

Cantilever moment

$$M_{boot} = q_{ult} \times l_{boot}^2/2 = \mathbf{4.0\ kNm/m}$$

Shear force

$$V_{boot} = q_{ult} \times l_{boot} = \mathbf{25.0\ kN/m}$$

K factor

$$K_{boot} = M_{boot}/(f_{cu} \times d_{boot}^2) = \mathbf{0.002}$$

Lever arm

$$Z_{boot} = d_{boot} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{boot}/0.9)}) = \mathbf{233\ mm}$$

Area of reinforcement required

$$A_{sboot\ req} = M_{boot}/((1.0/\gamma_s) \times f_{yboot} \times Z_{boot}) = \mathbf{40\ mm^2/m}$$

PASS - $A_{sboot\ req} \leq A_{sboot}$ - Area of reinforcement provided in boot is adequate for bending

Applied shear stress

$$V_{boot} = V_{boot}/d_{boot} = \mathbf{0.102\ N/mm^2}$$

Tension steel ratio

$$\rho_{boot} = 100 \times A_{sboot}/d_{boot} = \mathbf{0.160}$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$V_{cboot} = \mathbf{0.454\ N/mm^2}$$

PASS - $V_{boot} \leq V_{cboot}$ - Shear capacity of the boot is adequate

Corner design checks

Basic loading

Corner load number 1

Load type

Line load in y direction

Dead load

$$W_{Dcorner1} = \mathbf{34.7\ kN/m}$$

Live load

$$W_{Lcorner1} = \mathbf{10.2\ kN/m}$$

Ultimate load

$$W_{ultcorner1} = 1.4 \times W_{Dcorner1} + 1.6 \times W_{Lcorner1} = \mathbf{64.8\ kN/m}$$

Centroid of load from outside face of raft

$$X_{corner1} = \mathbf{175\ mm}$$

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Corner load number 2

Load type

Dead load

Live load

Ultimate load

Centroid of load from outside face of raft

Line load in x direction

$$W_{Dcorner2} = 28.7 \text{ kN/m}$$

$$W_{Lcorner2} = 6.3 \text{ kN/m}$$

$$W_{ultcorner2} = 1.4 \times W_{Dcorner2} + 1.6 \times W_{Lcorner2} = 50.3 \text{ kN/m}$$

$$y_{corner2} = 175 \text{ mm}$$

Corner bearing pressure check

Total uniform load at formation level

$$W_{udlcorner} = W_{Dudl} + W_{Ludl} + W_{edge}/b_{bearing} + W_{hcorethick} = 18.3 \text{ kN/m}^2$$

Net bearing press avail to resist line/point loads

$$q_{netcorner} = q_{allow} - W_{udlcorner} = 31.7 \text{ kN/m}^2$$

Total line/point loads

Total unfactored line load in x direction

$$W_{\Sigma line x} = 35.1 \text{ kN/m}$$

Total ultimate line load in x direction

$$W_{\Sigma ultline x} = 50.3 \text{ kN/m}$$

Total unfactored line load in y direction

$$W_{\Sigma line y} = 44.9 \text{ kN/m}$$

Total ultimate line load in y direction

$$W_{\Sigma ultline y} = 64.8 \text{ kN/m}$$

Total unfactored point load

$$W_{\Sigma point} = 0.0 \text{ kN}$$

Total ultimate point load

$$W_{\Sigma ultpoint} = 0.0 \text{ kN}$$

Length of side of sq reqd to resist line/point loads

$$p_{corner} = [W_{\Sigma line x} + W_{\Sigma line y} + \sqrt{(W_{\Sigma line x} + W_{\Sigma line y})^2 + 4 \times q_{netcorner} \times W_{\Sigma point}}] / (2 \times q_{netcorner})$$

$$p_{corner} = 2517 \text{ mm}$$

Bending moment about x-axis due to load/reaction eccentricity

Moment due to load 2 (x line)

$$M_{x2} = \max(0 \text{ kNm}, W_{ultcorner2} \times p_{corner} \times (p_{corner}/2 - y_{corner2})) = 137.3 \text{ kNm}$$

Total moment about x axis

$$M_{\Sigma x} = 137.3 \text{ kNm}$$

Bending moment about y-axis due to load/reaction eccentricity

Moment due to load 1 (y line)

$$M_{y1} = \max(0 \text{ kNm}, W_{ultcorner1} \times p_{corner} \times (p_{corner}/2 - x_{corner1})) = 176.9 \text{ kNm}$$

Total moment about y axis

$$M_{\Sigma y} = 176.9 \text{ kNm}$$

Check top reinforcement in edge beams for load/reaction eccentric moment

Max moment due to load/reaction eccentricity

$$M_{\Sigma} = \max(M_{\Sigma x}, M_{\Sigma y}) = 176.9 \text{ kNm}$$

Assume all of this moment is resisted by edge beam

From edge beam design checks away from corners

Moment due to edge beam spanning depression

$$M_{\Sigma edge} = 100.0 \text{ kNm}$$

Total moment to be resisted

$$M_{\Sigma cornerbp} = M_{\Sigma} + M_{\Sigma edge} = 276.9 \text{ kNm}$$

Width of section in compression zone

$$b_{edgetop} = b_{edge} + b_{boot} = 550 \text{ mm}$$

K factor

$$K_{cornerbp} = M_{\Sigma cornerbp} / (f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = 0.043$$

Lever arm

$$z_{cornerbp} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{cornerbp}/0.9)}) = 513 \text{ mm}$$

Total area of top steel required

$$A_{s cornerbp} = M_{\Sigma cornerbp} / ((1.0/\gamma_s) \times f_y \times z_{cornerbp}) = 1242 \text{ mm}^2$$

PASS - $A_{s cornerbp} \leq A_{s edgetop}$ - Area of reinforcement provided to resist eccentric moment is adequate

The allowable bearing pressure at the corner will not be exceeded

Corner beam bending check

Cantilever span of edge beam

$$l_{corner} = \phi_{depth}/\sqrt{(2)} + d_{edgetop}/2 = 2285 \text{ mm}$$

Moment and shear due to self weight

Ultimate self weight udl

$$W_{edgeult} = 1.4 \times W_{edge} = 12.1 \text{ kN/m}$$

Average ultimate slab udl (approx)

$$W_{cornerslab} = \max(0 \text{ kN/m}, 1.4 \times W_{slab} \times (\phi_{depth}/(\sqrt{(2)} \times 2) - (b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}))))$$

$$W_{cornerslab} = 3.3 \text{ kN/m}$$

Self weight and slab bending moment

$$M_{cornersw} = (W_{edgeult} + W_{cornerslab}) \times l_{corner}^2/2 = 40.2 \text{ kNm}$$

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Self weight and slab shear force

$$V_{\text{cornersw}} = (W_{\text{edgeult}} + W_{\text{cornerslab}}) \times l_{\text{corner}} = \mathbf{35.2 \text{ kN}}$$

Moment and shear due to udl

Maximum ultimate udl

$$W_{\text{cornerudl}} = ((1.4 \times W_{\text{Dudl}}) + (1.6 \times W_{\text{Ludl}})) \times \phi_{\text{depth}} / \sqrt{2} = \mathbf{11.9 \text{ kN/m}}$$

Bending moment

$$M_{\text{cornerudl}} = W_{\text{cornerudl}} \times l_{\text{corner}}^2 / 6 = \mathbf{10.3 \text{ kNm}}$$

Shear force

$$V_{\text{cornerudl}} = W_{\text{cornerudl}} \times l_{\text{corner}} / 2 = \mathbf{13.6 \text{ kN}}$$

Moment and shear due to line loads in x direction

Bending moment

$$M_{\text{cornerlinex}} = W_{\Sigma \text{ultlinex}} \times l_{\text{corner}}^2 / 2 = \mathbf{131.4 \text{ kNm}}$$

Shear force

$$V_{\text{cornerlinex}} = W_{\Sigma \text{ultlinex}} \times l_{\text{corner}} = \mathbf{115.0 \text{ kN}}$$

Moment and shear due to line loads in y direction

Bending moment

$$M_{\text{cornerliney}} = W_{\Sigma \text{ultliney}} \times l_{\text{corner}}^2 / 2 = \mathbf{169.3 \text{ kNm}}$$

Shear force

$$V_{\text{cornerliney}} = W_{\Sigma \text{ultliney}} \times l_{\text{corner}} = \mathbf{148.2 \text{ kN}}$$

Total moments and shears due to point loads

Bending moment about x axis

$$M_{\text{cornerpointx}} = \mathbf{0.0 \text{ kNm}}$$

Bending moment about y axis

$$M_{\text{cornerpointy}} = \mathbf{0.0 \text{ kNm}}$$

Shear force

$$V_{\text{cornerpoint}} = \mathbf{0.0 \text{ kN}}$$

Resultant moments and shears

Total moment about x axis

$$M_{\Sigma \text{cornerx}} = M_{\text{cornersw}} + M_{\text{cornerudl}} + M_{\text{cornerliney}} + M_{\text{cornerpointx}} = \mathbf{219.9 \text{ kNm}}$$

Total shear force about x axis

$$V_{\Sigma \text{cornerx}} = V_{\text{cornersw}} + V_{\text{cornerudl}} + V_{\text{cornerliney}} + V_{\text{cornerpoint}} = \mathbf{197.0 \text{ kN}}$$

Total moment about y axis

$$M_{\Sigma \text{cornery}} = M_{\text{cornersw}} + M_{\text{cornerudl}} + M_{\text{cornerlinex}} + M_{\text{cornerpointy}} = \mathbf{182.0 \text{ kNm}}$$

Total shear force about y axis

$$V_{\Sigma \text{cornery}} = V_{\text{cornersw}} + V_{\text{cornerudl}} + V_{\text{cornerlinex}} + V_{\text{cornerpoint}} = \mathbf{163.8 \text{ kN}}$$

Deflection of both edge beams at corner will be the same therefore design for average of these moments and shears

Design bending moment

$$M_{\Sigma \text{corner}} = (M_{\Sigma \text{cornerx}} + M_{\Sigma \text{cornery}}) / 2 = \mathbf{201.0 \text{ kNm}}$$

Design shear force

$$V_{\Sigma \text{corner}} = (V_{\Sigma \text{cornerx}} + V_{\Sigma \text{cornery}}) / 2 = \mathbf{180.4 \text{ kN}}$$

Reinforcement required in top of edge beam

K factor

$$K_{\text{corner}} = M_{\Sigma \text{corner}} / (f_{\text{cu}} \times b_{\text{edgetop}} \times d_{\text{edgetop}}^2) = \mathbf{0.031}$$

Lever arm

$$Z_{\text{corner}} = d_{\text{edgetop}} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{\text{corner}} / 0.9)}) = \mathbf{513 \text{ mm}}$$

Area of steel required for bending

$$A_{\text{scornerbend}} = M_{\Sigma \text{corner}} / ((1.0 / \gamma_s) \times f_y \times Z_{\text{corner}}) = \mathbf{901 \text{ mm}^2}$$

Minimum area of steel required

$$A_{\text{scornermin}} = A_{\text{sedgetopmin}} = \mathbf{408 \text{ mm}^2}$$

Area of steel required

$$A_{\text{scorner}} = \max(A_{\text{scornerbend}}, A_{\text{scornermin}}) = \mathbf{901 \text{ mm}^2}$$

PASS - $A_{\text{scorner}} \leq A_{\text{sedgetop}}$ - Area of reinforcement provided in top of edge beams at corners is adequate

Corner beam shear check

Average web width

$$b_w = b_{\text{edge}} + (h_{\text{edge}} / \tan(\alpha_{\text{edge}})) / 2 = \mathbf{523 \text{ mm}}$$

Applied shear stress

$$v_{\text{corner}} = V_{\Sigma \text{corner}} / (b_w \times d_{\text{edgetop}}) = \mathbf{0.639 \text{ N/mm}^2}$$

Tension steel ratio

$$\rho_{\text{corner}} = 100 \times A_{\text{sedgetop}} / (b_w \times d_{\text{edgetop}}) = \mathbf{0.556}$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$V_{\text{ccorner}} = \mathbf{0.564 \text{ N/mm}^2}$$

$V_{\text{corner}} \leq V_{\text{ccorner}} + 0.4 \text{ N/mm}^2$ - Therefore minimum links required

Link area to spacing ratio required

$$A_{\text{sv_upon_svreqcorner}} = 0.4 \text{ N/mm}^2 \times b_w / ((1.0 / \gamma_s) \times f_{ys}) = \mathbf{0.481 \text{ mm}}$$

Link area to spacing ratio provided

$$A_{\text{sv_upon_svprovedge}} = N_{\text{edgelink}} \times \pi \times \phi_{\text{edgelink}}^2 / (4 \times s_{\text{vedge}}) = \mathbf{0.785 \text{ mm}}$$

PASS - $A_{\text{sv_upon_svreqcorner}} \leq A_{\text{sv_upon_svprovedge}}$ - Shear reinforcement provided in edge beams at corners is adequate

Corner beam deflection check

Basic allowable span to depth ratio

$$\text{Ratio}_{\text{basiccorner}} = \mathbf{7.0}$$

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Moment factor	$M_{factorcorner} = M_{\Sigma corner} / (b_{edgetop} \times d_{edgetop}^2) = 1.253 \text{ N/mm}^2$
Steel service stress	$f_{scorner} = 2/3 \times f_y \times A_{scornerbend} / A_{sedgetop} = 191.201 \text{ N/mm}^2$
Modification factor	$MF_{corner} = \min(2.0, 0.55 + [(477 \text{ N/mm}^2 - f_{scorner}) / (120 \times (0.9 \text{ N/mm}^2 + M_{factorcorner})])$ $MF_{corner} = 1.656$
Modified allowable span to depth ratio	$Ratio_{allowcorner} = Ratio_{basiccorner} \times MF_{corner} = 11.593$
Actual span to depth ratio	$Ratio_{actualcorner} = l_{corner} / d_{edgetop} = 4.232$
PASS - $Ratio_{actualcorner} \leq Ratio_{allowcorner}$ - Edge beam span to depth ratio is adequate	

Internal beam design checks

Basic loading

Hardcore	$W_{hcorethick} = \gamma_{hcore} \times h_{hcorethick} = 3.0 \text{ kN/m}^2$
Internal beam self weight	$W_{int} = 24 \text{ kN/m}^3 \times [(h_{int} \times b_{int}) + (h_{int} - h_{slab})^2 / \tan(\alpha_{int}) + 2 \times h_{slab} \times (h_{int} - h_{slab}) / \tan(\alpha_{int})]$ $W_{int} = 11.5 \text{ kN/m}$

Internal beam load number 1

Load type	Longitudinal line load
Dead load	$W_{Dint1} = 54.4 \text{ kN/m}$
Live load	$W_{Lint1} = 19.2 \text{ kN/m}$
Ultimate load	$W_{ultint1} = 1.4 \times W_{Dint1} + 1.6 \times W_{Lint1} = 106.8 \text{ kN/m}$
Longitudinal line load width	$b_{int1} = 300 \text{ mm}$
Centroid of load from centreline of beam	$x_{int1} = 150 \text{ mm}$

Internal beam bearing pressure check

Total uniform load at formation level	$W_{udint} = W_{Dud1} + W_{Lud1} + W_{hcorethick} + 24 \text{ kN/m}^3 \times h_{int} = 21.4 \text{ kN/m}^2$
Sum of factored longitudinal and effective line loads	$\Sigma UDL_{int} = 106.8 \text{ kN/m}$
Sum of unfactored longitudinal and effective line loads	$\Sigma UDL_{lsint} = 73.6 \text{ kN/m}$

Centroid of loads from centreline of internal beam

Load x distance for internal load 1	$Moment_{int1} = W_{ultint1} \times x_{int1} = 16.0 \text{ kN}$
Sum of load x distances	$\Sigma Moment_{int} = 16.0 \text{ kN}$
Centroid of loads	$x_{barint} = \Sigma Moment_{int} / \Sigma UDL_{int} = 150.0 \text{ mm}$
Moment due to eccentricity to be resisted by slab	$M_{eccint} = \Sigma UDL_{int} \times \text{abs}(x_{barint}) = 16.0 \text{ kNm/m}$
Assume moment due to eccentricity is resisted equally by top steel of slab on one side and bottom steel of slab on other	
From slab bending check	

Moment due to depression under slab (hogging)	$M_{\Sigma e} = 3.9 \text{ kNm/m}$
Total moment to be resisted by slab top steel	$M_{slabtopint} = M_{eccint} / 2 + M_{\Sigma e} = 11.9 \text{ kNm/m}$
K factor	$K_{slabtopint} = M_{slabtopint} / (f_{cu} \times d_{slabmin}^2) = 0.009$
Lever arm	$Z_{slabtopint} = d_{slabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtopint} / 0.9)}) = 171 \text{ mm}$
Area of steel required	$A_{sslabtopintreq} = M_{slabtopint} / ((1.0 / \gamma_s) \times f_{yslab} \times Z_{slabtopint}) = 160 \text{ mm}^2/\text{m}$

PASS - $A_{sslabtopintreq} \leq A_{sslabtop}$ - Area of reinforcement in top of slab is adequate to transfer moment into slab

Mt to be resisted by slab btm stl due to load ecc'ty	$M_{slabbtmint} = M_{eccint} / 2 = 8.0 \text{ kNm/m}$
K factor	$K_{slabbtmint} = M_{slabbtmint} / (f_{cu} \times d_{slabmin}^2) = 0.008$
Lever arm	$Z_{slabbtmint} = d_{slabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbtmint} / 0.9)}) = 152 \text{ mm}$
Area of steel required in bottom	$A_{sslabbtmintreq} = M_{slabbtmint} / ((1.0 / \gamma_s) \times f_{yslab} \times Z_{slabbtmint}) = 121 \text{ mm}^2/\text{m}$

PASS - $A_{sslabbtmintreq} \leq A_{sslabbtm}$ - Area of reinforcement in bottom of slab is adequate to transfer moment into slab

Bearing pressure

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Initially check bearing pressure based on beam soffit/chamfer width only

Allowable bearing width	$b_{\text{bearing}} = \min(2 \times (b_{\text{int}}/2 \times \tan(\alpha_{\text{int}}) + h_{\text{int}})/(1 + \tan(\alpha_{\text{int}})), b_{\text{int}} + 2 \times (h_{\text{int}} - h_{\text{slab}})/\tan(\alpha_{\text{int}}))$ $b_{\text{bearing}} = 756 \text{ mm}$
Bearing pressure due to line/point loads	$q_{\text{linepointint}} = \Sigma \text{UDLs}_{\text{int}}/b_{\text{bearing}} = 97.3 \text{ kN/m}^2$
Total applied bearing pressure	$q_{\text{int}} = q_{\text{linepointint}} + w_{\text{udlint}} = 118.7 \text{ kN/m}^2$ $q_{\text{int}} > q_{\text{allow}}$ - The slab is required to resist some of the beam loading

Now consider slab to resist some of beam load

Bearing width required	$b_{\text{reqint}} = \Sigma \text{UDLs}_{\text{int}}/(q_{\text{allow}} - w_{\text{udlint}}) = 2572 \text{ mm}$
Effective bearing width at u/s of beam/slab	$b_{\text{reqeffint}} = b_{\text{reqint}} - 2 \times h_{\text{hcorethick}} \times \tan(30) = 2399 \text{ mm}$
Resultant ult upward press acting on u/s of slab	$q_{\text{result}} = (q_{\text{allow}} - w_{\text{udlint}}) \times b_{\text{reqint}}/b_{\text{reqeffint}} \times \Sigma \text{UDL}_{\text{int}}/\Sigma \text{UDLs}_{\text{int}} = 44.5 \text{ kN/m}^2$
Slab cantilever moment	$M_{\text{cantint}} = q_{\text{result}} \times [(b_{\text{reqint}} - b_{\text{int}})/2]^2/2 = 23.9 \text{ kNm/m}$
Total moment to be resisted by slab	$M_{\text{slabint}} = M_{\text{cantint}} + M_{\text{eccint}}/2 = 31.9 \text{ kNm/m}$
K factor	$K_{\text{slabint}} = M_{\text{slabint}}/(f_{\text{cu}} \times d_{\text{bslabmin}}^2) = 0.031$
Lever arm	$Z_{\text{slabint}} = d_{\text{bslabmin}} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{\text{slabint}}/0.9)}) = 152 \text{ mm}$
Area of steel required	$A_{\text{sslabint}} = M_{\text{slabint}}/((1.0/\gamma_s) \times f_{\text{yslab}} \times Z_{\text{slabint}}) = 483 \text{ mm}^2/\text{m}$ PASS - $A_{\text{sslabint}} \leq A_{\text{sslabbtm}}$ - Area of reinforcement provided to distribute the beam load is adequate The allowable bearing pressure under the internal beam will not be exceeded

Internal beam bending check

Divider for moments due to udl's	$\beta_{\text{udl}} = 10.0$
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Applied bending moments

Span of internal beam	$l_{\text{int}} = \phi_{\text{depththick}} + d_{\text{inttop}} = 3392 \text{ mm}$
Ultimate self weight udl	$w_{\text{intult}} = 1.4 \times w_{\text{int}} = 16.1 \text{ kN/m}$
Ultimate slab udl (approx)	$w_{\text{intslab}} = \max(0 \text{ kN/m}, 1.4 \times w_{\text{slab}} \times ((\phi_{\text{depththick}} \times 3/4) - (b_{\text{int}} + 2 \times (h_{\text{int}} - h_{\text{slab}})/\tan(\alpha_{\text{int}}))))$ $w_{\text{intslab}} = 9.1 \text{ kN/m}$
Self weight and slab bending moment	$M_{\text{intsw}} = (w_{\text{intult}} + w_{\text{intslab}}) \times l_{\text{int}}^2/\beta_{\text{udl}} = 29.0 \text{ kNm}$
Self weight shear force	$V_{\text{intsw}} = (w_{\text{intult}} + w_{\text{intslab}}) \times l_{\text{int}}/2 = 42.7 \text{ kN}$

Moments due to applied uniformly distributed loads

Ultimate udl (approx)	$w_{\text{intudl}} = w_{\text{udlult}} \times \phi_{\text{depththick}} \times 3/4 = 12.6 \text{ kN/m}$
Bending moment	$M_{\text{intudl}} = w_{\text{intudl}} \times l_{\text{int}}^2/\beta_{\text{udl}} = 14.5 \text{ kNm}$
Shear force	$V_{\text{intudl}} = w_{\text{intudl}} \times l_{\text{int}}/2 = 21.4 \text{ kN}$

Moment and shear due to load number 1

Bending moment	$M_{\text{int1}} = w_{\text{ultint1}} \times l_{\text{int}}^2/\beta_{\text{udl}} = 122.9 \text{ kNm}$
Shear force	$V_{\text{int1}} = w_{\text{ultint1}} \times l_{\text{int}}/2 = 181.2 \text{ kN}$

Resultant moments and shears

Total moment (hogging and sagging)	$M_{\Sigma \text{int}} = 166.4 \text{ kNm}$
Maximum shear force	$V_{\Sigma \text{int}} = 245.3 \text{ kN}$

Reinforcement required in top

Width of section in compression zone	$b_{\text{inttop}} = b_{\text{int}} = 500 \text{ mm}$
Average web width	$b_{\text{wint}} = b_{\text{int}} + h_{\text{int}}/\tan(\alpha_{\text{int}}) = 846 \text{ mm}$
K factor	$K_{\text{inttop}} = M_{\Sigma \text{int}}/(f_{\text{cu}} \times b_{\text{inttop}} \times d_{\text{inttop}}^2) = 0.028$
Lever arm	$Z_{\text{inttop}} = d_{\text{inttop}} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{\text{inttop}}/0.9)}) = 515 \text{ mm}$

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Area of steel required for bending

$$A_{sinttopbend} = M_{\Sigma int} / ((1.0/\gamma_s) \times f_y \times Z_{inttop}) = \mathbf{743 \text{ mm}^2}$$

Minimum area of steel

$$A_{sinttopmin} = 0.0013 \times b_{wint} \times h_{int} = \mathbf{660 \text{ mm}^2}$$

Area of steel required

$$A_{sinttopreq} = \max(A_{sinttopbend}, A_{sinttopmin}) = \mathbf{743 \text{ mm}^2}$$

PASS - $A_{sinttopreq} \leq A_{sinttop}$ - Area of reinforcement provided in top of internal beams is adequate

Reinforcement required in bottom

Width of section in compression zone

$$b_{intbtm} = b_{int} + 2 \times (h_{int} - h_{slab}) / \tan(\alpha_{int}) + 0.2 \times l_{int} = \mathbf{1611 \text{ mm}}$$

K factor

$$K_{intbtm} = M_{\Sigma int} / (f_{cu} \times b_{intbtm} \times d_{intbtm}^2) = \mathbf{0.009}$$

Lever arm

$$Z_{intbtm} = d_{intbtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{intbtm}/0.9)}) = \mathbf{505 \text{ mm}}$$

Area of steel required for bending

$$A_{sintbtmbend} = M_{\Sigma int} / ((1.0/\gamma_s) \times f_y \times Z_{intbtm}) = \mathbf{757 \text{ mm}^2}$$

Minimum area of steel required

$$A_{sintbtmmin} = 0.0013 \times 1.0 \times b_{wint} \times h_{int} = \mathbf{660 \text{ mm}^2}$$

Area of steel required

$$A_{sintbtmreq} = \max(A_{sintbtmbend}, A_{sintbtmmin}) = \mathbf{757 \text{ mm}^2}$$

PASS - $A_{sintbtmreq} \leq A_{sintbtm}$ - Area of reinforcement provided in bottom of internal beams is adequate

Internal beam shear check

Applied shear stress

$$V_{int} = V_{\Sigma int} / (b_{wint} \times d_{inttop}) = \mathbf{0.535 \text{ N/mm}^2}$$

Tension steel ratio

$$\rho_{int} = 100 \times A_{sinttop} / (b_{wint} \times d_{inttop}) = \mathbf{0.175}$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$V_{cint} = \mathbf{0.383 \text{ N/mm}^2}$$

$V_{int} \leq V_{cint} + 0.4 \text{ N/mm}^2$ - Therefore minimum links required

Link area to spacing ratio required

$$A_{sv_upon_svreqint} = 0.4 \text{ N/mm}^2 \times b_{wint} / ((1.0/\gamma_s) \times f_{ys}) = \mathbf{0.779 \text{ mm}}$$

Link area to spacing ratio provided

$$A_{sv_upon_svprovint} = N_{intlink} \times \pi \times \phi_{intlink}^2 / (4 \times S_{vint}) = \mathbf{0.785 \text{ mm}}$$

PASS - $A_{sv_upon_svreqint} \leq A_{sv_upon_svprovint}$ - Shear reinforcement provided in internal beams is adequate

GENERAL NOTES:

These notes apply to the entire drawing set.

Dimensions: All dimensions are in millimeters and levels are in meters unless noted otherwise. Do not scale from drawings; use figured dimensions only.

Discrepancies: The Contractor must verify all dimensions and levels on-site prior to commencement of work. Any discrepancies between structural, architectural, and services drawings must be reported to the Engineer immediately for clarification.

Standards: All workmanship and materials are to be in strict accordance with the latest relevant local Building Codes and Standards.

Temporary Stability: The Contractor is solely responsible for the stability of the structure during construction, including all necessary temporary bracing and shoring.

Safety: All construction operations must adhere to current Health & Safety regulations.

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

Party wall agreement: According to the law, any construction work that interferes with the shared wall (party wall), including beams resting on the shared wall, and works near the shared wall should be subject to a party wall agreement.
<https://www.gov.uk/party-walls-building-works>

Installing Temporary Supports:

Acrow Props and Strongboys: Above the opening, remove small sections of plaster to reveal brick/block joints. Install strongboys into the bed joints above the proposed beam position. Place Acrow props underneath the strongboys at regular intervals (usually every 900–1200mm depending on wall and span). Ensure props are plumb and on solid base plates or timber spreaders.

Double-Sided Support (if required): For cavity or thicker walls, support both skins using back-to-back strongboys or needle beams with Acrows on both sides.

STRUCTURAL STEELWORK NOTES (BEAMS & COLUMNS):

Focuses on steel grade, connections, and protection.

Steel Grade: All structural steel to be Grade S275 (JR/J0/J2) unless noted otherwise. Hollow sections to be S355 J2H.

Fabrication: Fabrication and erection to comply with relevant execution standards (e.g., EN 1090 or local equivalent).

Welding: All welding to be carried out by certified welders using full-strength continuous fillet welds (minimum 6mm leg length) unless detailed otherwise.

Bolting:

- Ordinary Bolts: Grade 8.8.
- High Strength Friction Grip (HSFG) Bolts: Grade 10.9 (if applicable).
- Washers to be provided under all nuts and bolt heads.

Beam Bearings: Steel beams resting on masonry must have concrete padstones (Grade C35) or spreader plates as detailed. Minimum bearing length 150mm.

Corrosion Protection:

- Internal Steel: Wire brush to remove rust/scale and apply one coat of zinc phosphate primer and two top coats.
- External/Embedded Steel: Hot-dip galvanized to [Standard Number] (min 85 microns).
- Encased Steel: Bituminous paint where in contact with concrete/masonry.

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

FOUNDATION & GEOTECHNICAL NOTES (RAFTS & FOOTINGS):

Essential for ground conditions and concrete interface.

Bearing Capacity: Foundations have been designed for a safe allowable ground bearing capacity of **50 kN/m²** unless geotechnical investigation says different. The Contractor must confirm this capacity on-site before pouring concrete.

Soil Inspection: All foundation excavations must be inspected and approved by the Building Control Officer or Engineer before pouring concrete.

Soft Spots: Any soft spots encountered at foundation level are to be excavated and replaced with mass concrete (Grade C15) or engineered fill as directed by the Engineer.

Blinding: Provide a minimum **50mm** thick blinding under all reinforced concrete foundations.

Groundwater: The Contractor shall keep excavations free from water. Pumping may be required.

Tree Roots: If tree roots are encountered in excavations, the Engineer must be consulted. Additional depth or compressible heave protection (e.g., Clayboard) may be required. **This is especially important in the cohesive soils (clay).**

REINFORCED CONCRETE NOTES (GENERAL & RAFTS):

Concrete Grade: Unless noted otherwise, structural concrete shall be **Grade C30/37** (Cylinder/Cube strength) with a minimum cement content of 325 kg/m³.

Aggregates: Maximum aggregate size to be **20mm**.

Reinforcement:

- High Yield Steel (Deformed Bars): Grade 500B ($f_y = 500\text{N/mm}^2$). Notation: 'H' or 'T'.
- Mild Steel (Links): Grade 250 ($f_y = 250\text{N/mm}^2$). Notation: 'R'.

Cover to Reinforcement (Minimum):

- Foundations/Earth faces: **50mm** (or 75mm if cast directly against earth without blinding).
- External columns/beams: **40mm**.
- Internal slabs/walls: **25mm**.

Laps & Anchorage: Lap lengths for reinforcement shall be **50 x Bar Diameter** unless detailed otherwise. Stagger laps where possible.

Spacer Blocks: Reinforcement must be rigidly fixed and supported by approved concrete or plastic spacers to ensure correct cover.

Curing: Concrete must be properly cured (kept moist and protected from frost/direct sun) for a minimum of **7 days** after placement.

Holes/Chases: No holes or chases are to be cut in structural concrete without prior approval from the Engineer.

RETAINING WALL NOTES:

Critical for water pressure management and stability.

Backfill: Do not backfill behind the wall until the concrete has achieved adequate strength (**minimum 28 days**) and the floor slab (propping element) is in place, unless the wall is designed as a free cantilever.

Backfill Material: Backfill to be free-draining granular material (e.g., crushed stone), compacted in layers not exceeding **225mm**. Do not use heavy vibratory equipment close to the wall.

Drainage: Provide a **100mm diameter perforated agricultural drain pipe** at the base of the wall (heel side), surrounded by clean gravel and wrapped in geotextile filter fabric. Connect to a suitable outfall.

Weep Holes: (If applicable) Provide 50mm diameter weep holes at **1200mm centers** horizontally and vertically along the wall face.

Waterproofing: Apply tanking membrane to the earth-retaining face of the wall in accordance with manufacturer instructions.

MASONRY NOTES (LOAD BEARING):

Brick/Block Strength:

- Load-bearing brickwork: Minimum compressive strength **20 N/mm²**.
- Blockwork: Minimum compressive strength **7.3 N/mm²** (solid blocks) unless noted otherwise.

Mortar: Mortar designation (e.g., 1:1:6 cement:lime:sand) or Class M4/M6.

Wall Ties: Stainless steel wall ties to be provided at **900mm horizontal** and **450mm vertical** centers (staggered), and at every course at movement joints/openings.

Damp Proof Course (DPC): DPC to be provided a minimum of **150mm** above external ground level.

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

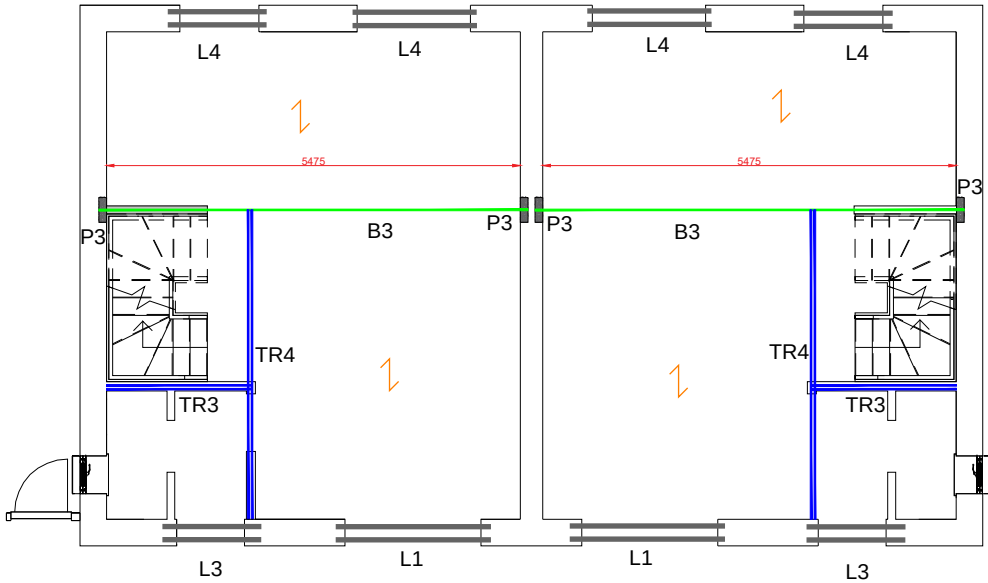
- NOTES:
1. ALL DIMENSIONS IN mm UNLESS NOTED OTHERWISE.
 2. DO NOT SCALE FROM THIS DRAWING.
 3. ALL DIMENSIONS TO BE VERIFIED ON SITE PRIOR TO COMMENCING WORKS

B3 = UB 254x146x37 S275

TR3= C24 2x47x225 @ 400mm C/C

TR3= C24 2x47x225 @ 400mm C/C + 8x225 steel plate

P3 = 330x100x140mm deep padstone



Ground floor plan shows structure above

Load Table		S4	R6	R9	R12
Units suitable for 100mm wide walls Unfactored Loads in kN/m					
Fire Resistance Available (mins)		30	30	30	30
Suitable For Foundation Use		Yes	Yes	Yes	Yes
Maximum Stock Length Available		3000mm	3000mm	3600mm Longer lengths available on request - up to 4800mm	3600mm
Available Range Finish		Standard/Faced	Standard/Faced	Standard/Faced	Standard/Faced
Length	Clear Span	100x100	100x140	100x215	100x290
900mm	700mm	15.77	38.73	96.25	127.77
1200mm	1000mm	10.90	30.87	62.92	83.26
1500mm	1200mm	9.41	27.22	50.00	77.43
1800mm	1500mm	7.21	19.32	37.15	63.07
2100mm	1800mm	5.04	13.59	28.63	47.94
2400mm	2100mm	3.70	10.04	22.61	36.95
2700mm	2400mm	2.81	7.70	18.13	28.62
3000mm	2700mm	2.19	6.06	14.48	22.77
3300mm	3000mm	n/a	4.88	11.68	18.51
3600mm	3200mm	n/a	4.27	9.78	15.79
Lintel Weight kg/m		23	34	52	70

NOTE: CONTRACTOR TO VERIFY OPENING DIMENSIONS ON SITE AND CONFIRM LINTEL LENGTH AND REQUIRED BEARING PRIOR TO ORDERING.



Client
Philip Harris (Arch) TBC

Project
1a Blaendowlais, Dowlais,
Merthyr Tydfil, CF48 3RB

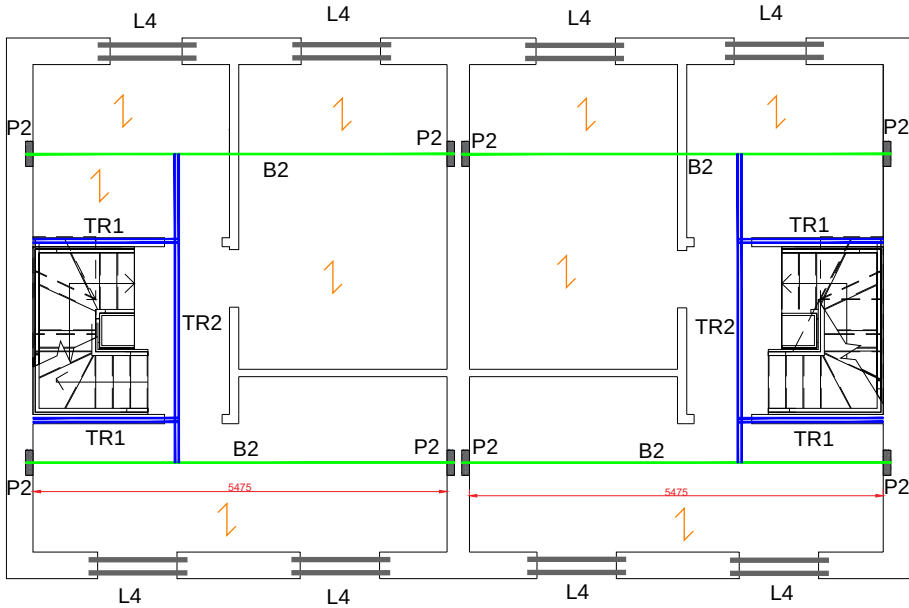
Drawing Title
Ground floor rear extension

Drawing Status
For Construction

Drawn By: MK	Date: 24/08/2026	Checked By: MK	Date:
Original Size A3	Scale 1:100	Issue Date	
Project Number AUX- 843	Drawing Number 101	Revision C01	

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

- NOTES:
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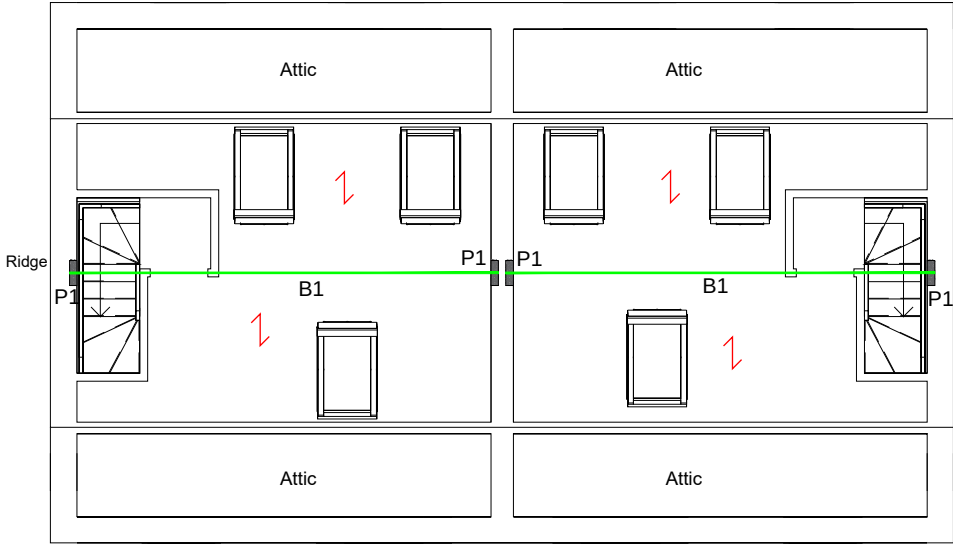
B2 = UB 254x146x37 S275
↔ Floor Joists = C24 47x225 @ 400mm C/C
TR1 = TR2= C24 2x47x225 @ 400mm C/C
P2 = 330x100x140mm deep padstone

First floor plan shows structure above

AUXILIUM STRUCTURAL ENGINEERING Mobile: +44 7465819500 Email: design@auxilium.com			
Client			
Philip Harris (Arch) TBC			
Project			
1a Blaendowlais, Dowlais, Merthyr Tydfil, CF48 3RB			
Drawing Title			
First floor plan shows structure above			
Drawing Status			
For Construction			
Drawn By:	Date:	Checked By:	Date:
MK	24/08/2026	MK	
Original Size	Scale	Issue Date	
A3	1:100		
Project Number	Drawing Number	Revision	
AUX- 843	102	C01	

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

- NOTES:
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B1 = UB 203x133x25 S275
↔ Rafters = C24 47x150 @ 600mm C/C
P1 = 330x100x140mm deep padstone

Second floor plan shows structure above

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Client			
Philip Harris (Arch) TBC			
Project			
1a Blaendowlais, Dowlais, Merthyr Tydfil, CF48 3RB			
Drawing Title			
Ground floor rear extension			
Drawing Status			
For Construction			
Drawn By:	Date:	Checked By:	Date:
MK	24/08/2026	MK	
Original Size	Scale	Issue Date	
A3	1:100		
Project Number	Drawing Number	Revision	
AUX- 843	103	C01	

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

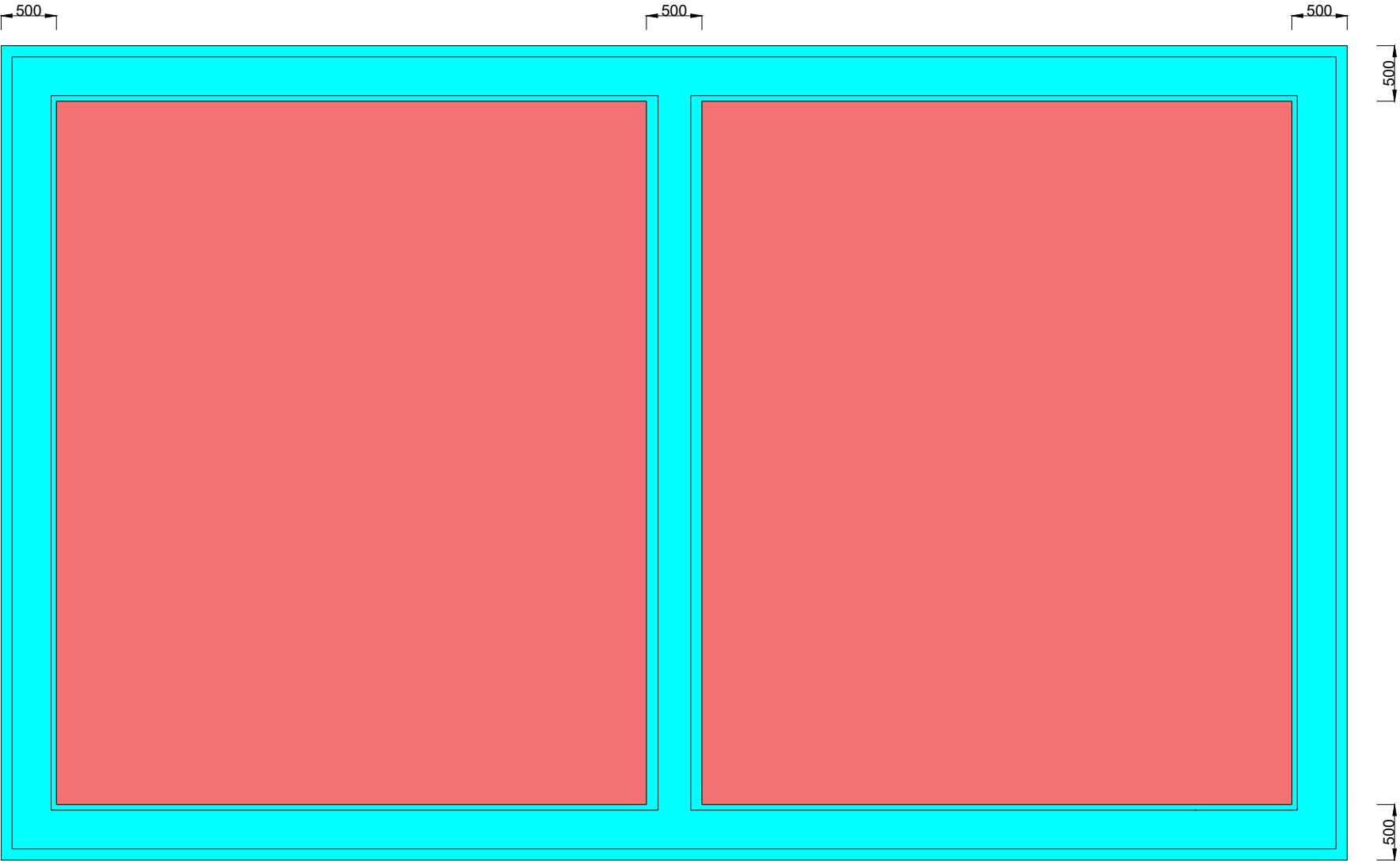
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Edge and internal beams
(See drawing 105 for reinforcement details)



225mm deep concrete slab with 2xA393 mesh at top and 2xA393 mesh at bottom
(See drawing 105 for reinforcement details)



Raft foundation plan



Client
Philip Harris (Arch) TBC

Project
1a Blaendowlais, Dowlais,
Merthyr Tydfil, CF48 3RB

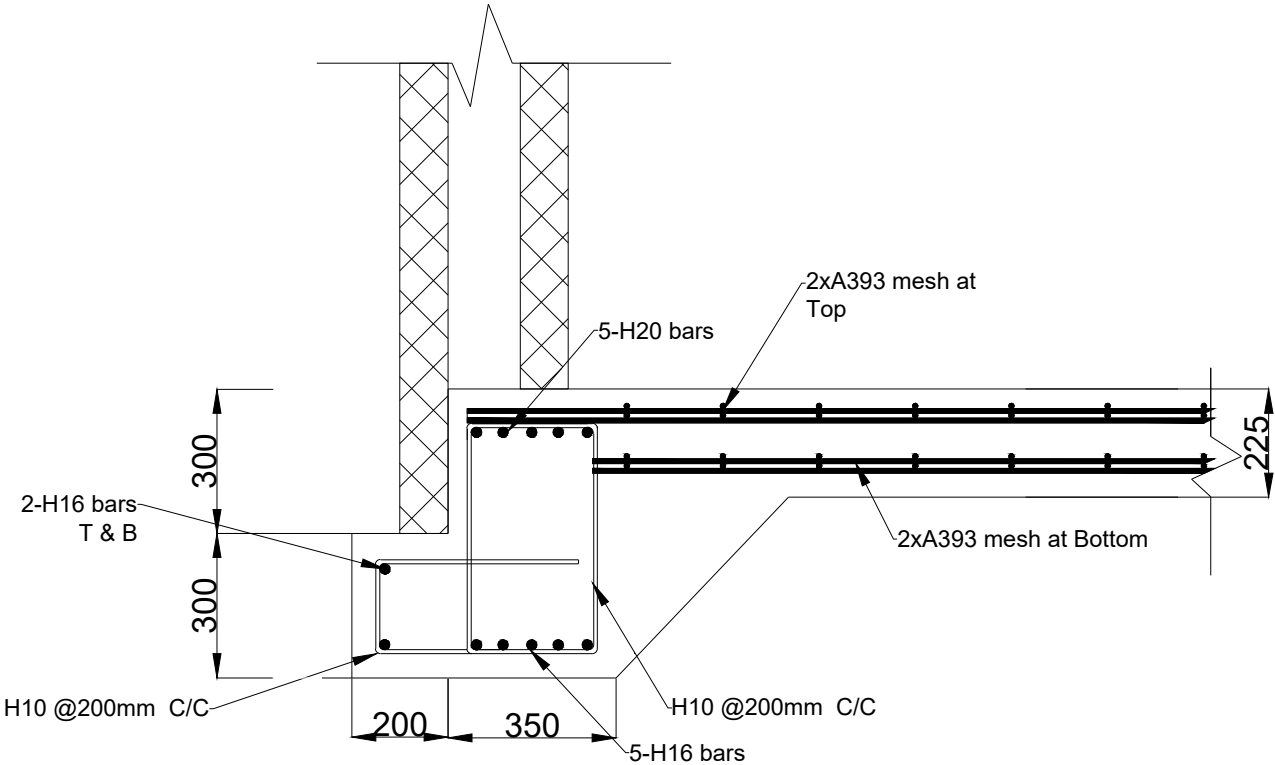
Drawing Title
Raft foundation plan

Drawing Status
For Construction

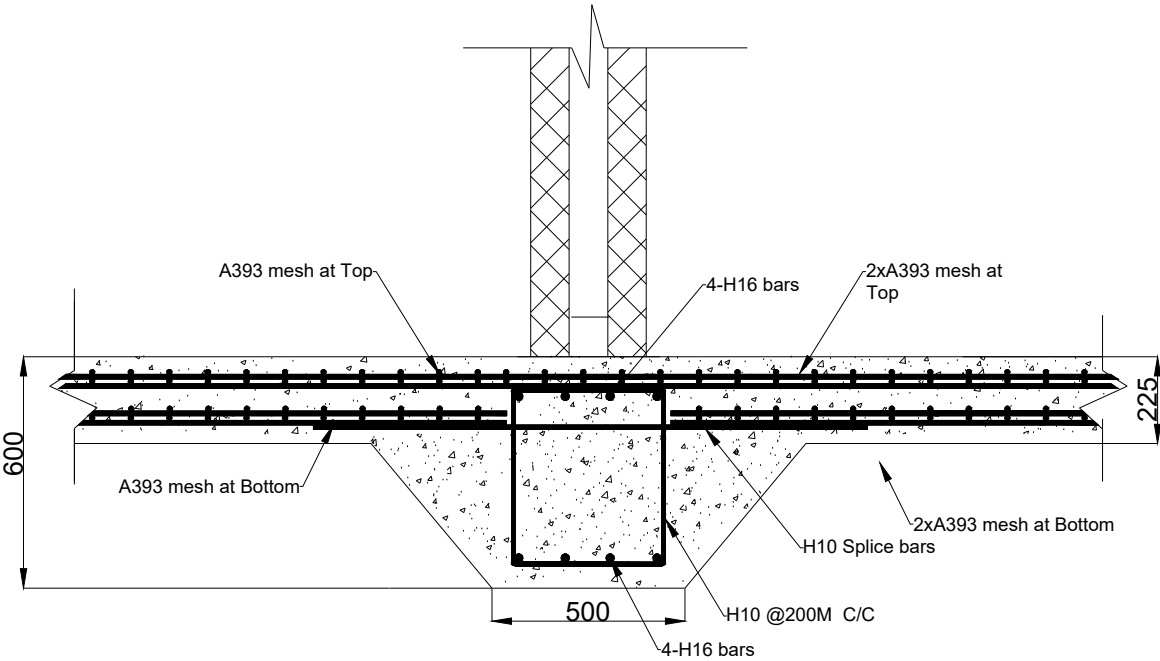
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Original Size A3	Scale 1:100	Issue Date	
Project Number AUX- 843	Drawing Number 104	Revision C01	

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

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External beam



Internal beam
Foundation sections



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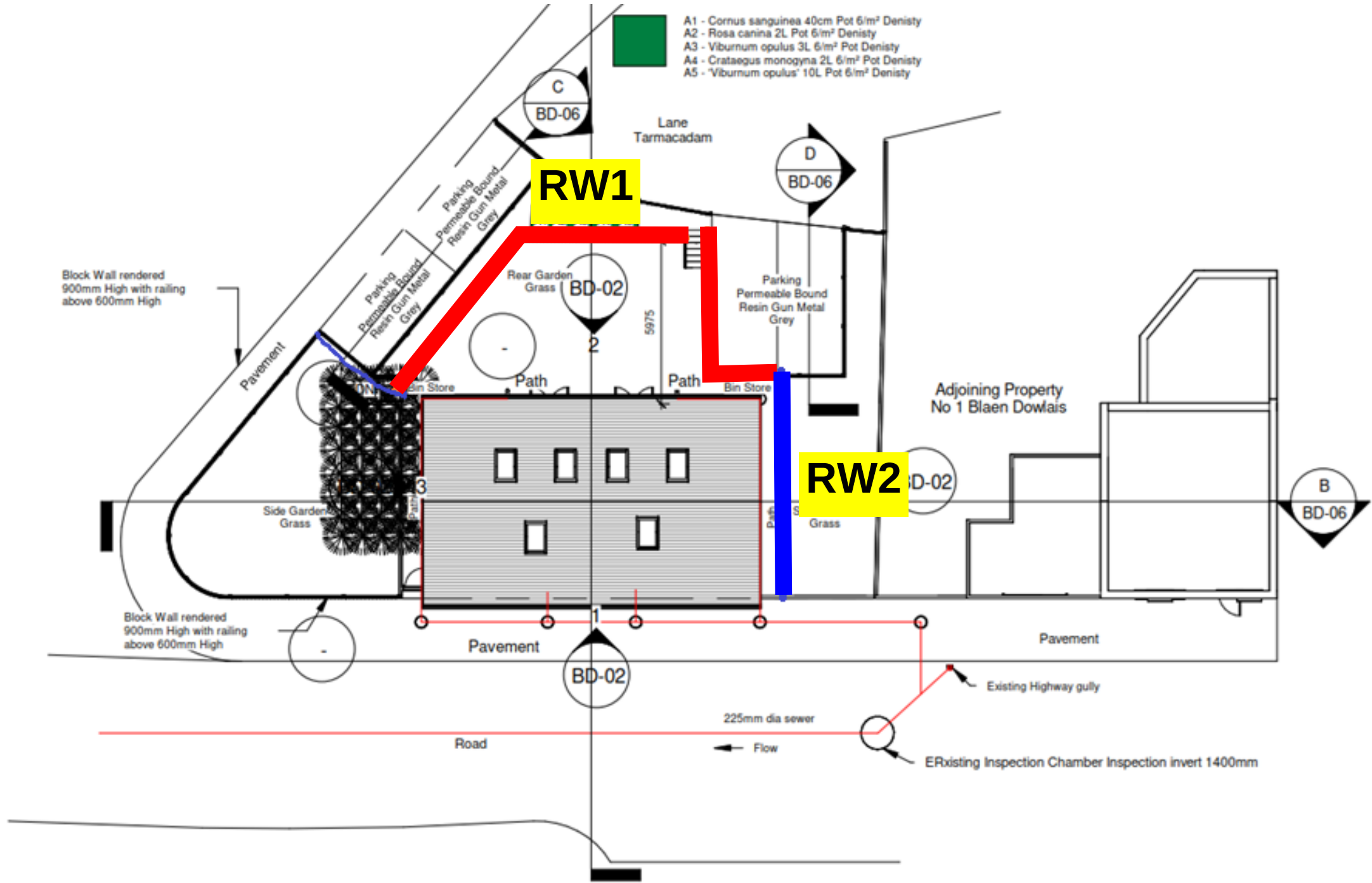
Drawing Title
Foundation sections

Drawing Status
For Construction

Drawn By: MK	Date: 24/08/2026	Checked By: MK	Date:
Original Size A3	Scale 1:100	Issue Date	
Project Number AUX- 843	Drawing Number 105	Revision C01	

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

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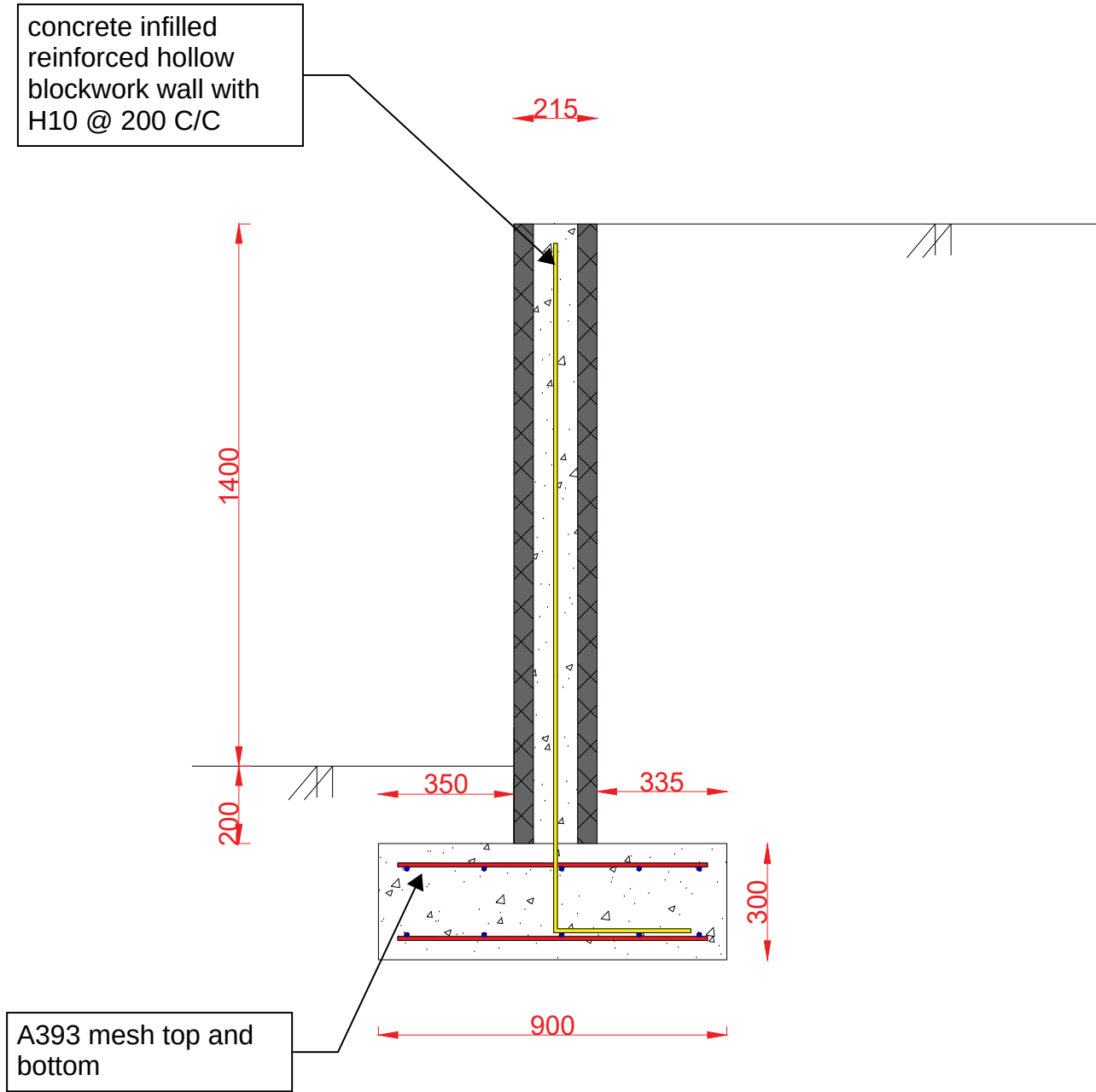
Plan showing retaining wall location



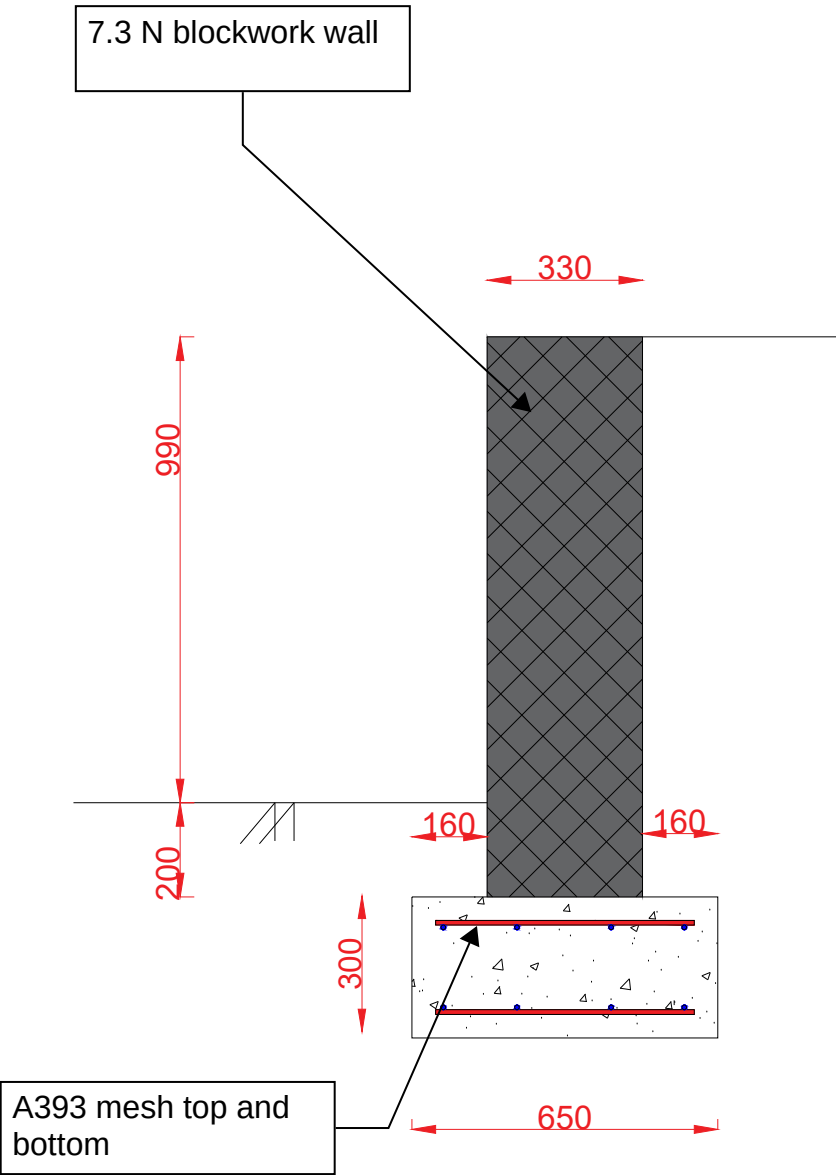
Client			
Philip Harris (Arch) TBC			
Project			
1a Blaendowlais, Dowlais, Merthyr Tydfil, CF48 3RB			
Drawing Title			
Plan showing retaining wall location			
Drawing Status			
For Construction			
Drawn By:	Date:	Checked By:	Date:
MK	24/08/2026	MK	
Original Size	Scale	Issue Date	
A3	NTS		
Project Number	Drawing Number	Revision	
AUX- 843	106	C01	

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

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RW1



RW2



Client
Philip Harris (Arch) TBC

Project
1a Blaendowlais, Dowlais,
Merthyr Tydfil, CF48 3RB

Drawing Title
Retaining wall section

Drawing Status
For Construction

Drawn By: MK	Date: 24/08/2026	Checked By: MK	Date:
Original Size A3	Scale NTS	Issue Date	
Project Number AUX- 843	Drawing Number 107	Revision C01	



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