

Vale Consultancy

CONSULTING CIVIL & STRUCTURAL ENGINEERS

STRUCTURAL CALCULATIONS

9 Pond Row,
Abercanaid,
Merthyr Tydfil,
CF48 1YT.

Prepared for: Scott Pritchard
Project Ref: B2098

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DOCUMENT CONTROL

Project:	9 Pond Row, Abercanaid, Merthyr Tydfil, CF48 1YT
Client:	Scott Pritchard
Vale Consultancy Ref:	B2098

Issue	Prepared by	Approved by	Date	Status
01	Harry Hodder BSc (Hons)	Chris Danielsen BEng (Hons) CEng MIStructE	10.06.2025	First issue



DESIGN STATEMENT

The design has been completed in accordance with British Standards BS6399, BS5628 (Masonry), BS5268 (Timber) & BS 5950 (Steel).

Information for this has been collated from survey/site visit and information provided by the client.

The design is for 1No. pair of beams to provide main support to the main knock-through on the rear elevation to be supported on steel column, 4No. timber trimmers to trim around the proposed staircase at both ground and first floor level and a raft foundation for the proposed rear extension.

The steel fabricator and contractor must check all site dimensions for the steel beams and posts prior to fabrication.

Ground bearing pressure assumed to be 50kPa, to be confirmed on site.

Contractor to check suitability of existing masonry for padstone and steel bearing.

Existing & proposed blockwork assumed to have minimum 7.3N/mm² compressive strength.

Refer to our sketches B2098 – SK01 and B2098 – S01 and for references.

All dimensions, weights, member lengths, member tables, and any other relevant design information contained within this package is for analysis, and design purposes only, for submission to building control, and must not be used for pricing or fabrication purposes. All queries are to be directed to the engineer prior to tendering, pricing or fabrication. All structural members dimensions need to be taken accurately on site by the contractor prior to fabrication.

During any initial site visits performed by our Engineer, assumptions have to be made where structural elements may have been obscured by finishes. Opening up works should be undertaken prior to fabrication or construction of any structural members to confirm that the structural layout as indicated in our proposed sketches/detailed drawings is correct.

If there are any discrepancies, the Engineer MUST be informed immediately, and prior to proceeding with construction works on site.

CDM Comments:

- All temporary propping/support is to correspond to contractor's specification and details.

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Proposed Rear Extension at 9 Pond Row, Abercanaid, Merthyr Tydfil

The design has been completed in accordance with British Standards BS6399, BS5628 (Masonry), BS5268 (Timber) & BS 5950 (Steel). All dimensions stated in the following calculations are for design purposes only and not for use for fabrication or construction purposes. All dimensions must be checked by a competent contractor prior to fabrication or construction and any discrepancies found are to be reported to the design engineer.

LOAD TAKEDOWN

Pitched Roof

Dead Loading:

Roof pitch (Max);	$\Theta_{PR} = 40^\circ$
Roof cover (Concrete Tiles);	$PRoof_{d1} = 0.55kN/m^2$
Insulation, felt and battens;	$PRoof_{d2} = 0.10kN/m^2$
Purlins and rafters;	$PRoof_{d3} = 0.20kN/m^2$
Ceiling;	$PRoof_{d4} = 0.15kN/m^2$
Total Roof Dead Load;	$PRoof_d = PRoof_{d1} + PRoof_{d2} + PRoof_{d3} + PRoof_{d4} / \sin(\Theta_{PR}) = 1.08kN/m^2$

Imposed Loading:

Snow;	$PRoof_{im1} = 0.60kN/m^2$
Total Roof Imposed Load;	$PRoof_{im} = PRoof_{im1} = 0.60kN/m^2$

Masonry Walls

Dead Loading:

Render;	$Wall_{d1} = 0.15kN/m^2$
450mm Masonry;	$Wall_{d2} = 22.00kN/m^3 \times 0.45m = 9.90kN/m^2$
Plaster;	$Wall_{d3} = 0.15kN/m^2$
Total Wall Dead Load;	$Wall_d = Wall_{d1} + Wall_{d2} + Wall_{d3} = 10.20kN/m^2$

First Floor

Dead Loading:

Timber Floor boards;	$FFloor_{d1} = 0.15kN/m^2$
Insulation;	$FFloor_{d2} = 0.05kN/m^2$
Timber Joists;	$FFloor_{d3} = 0.15kN/m^2$
Ceiling;	$FFloor_{d4} = 0.15kN/m^2$
Total First Floor Dead Load;	$FFloor_d = FFloor_{d1} + FFloor_{d2} + FFloor_{d3} + FFloor_{d4} = 0.50kN/m^2$

Imposed Loading:

Residential;	$FFloor_{im1} = 1.50kN/m^2$
Total First Floor Imposed Load;	$FFloor_{im} = FFloor_{im1} = 1.50kN/m^2$

TIMBER TRIMMER T1 - DESIGN

Design Data:

Design Length;

 $L_{des} = 3.00m$

Loaded Width for floor;

 $L_{w_ff} = 3.00m / 2 = 1.50m$

Dead Load Applied to trimmer;

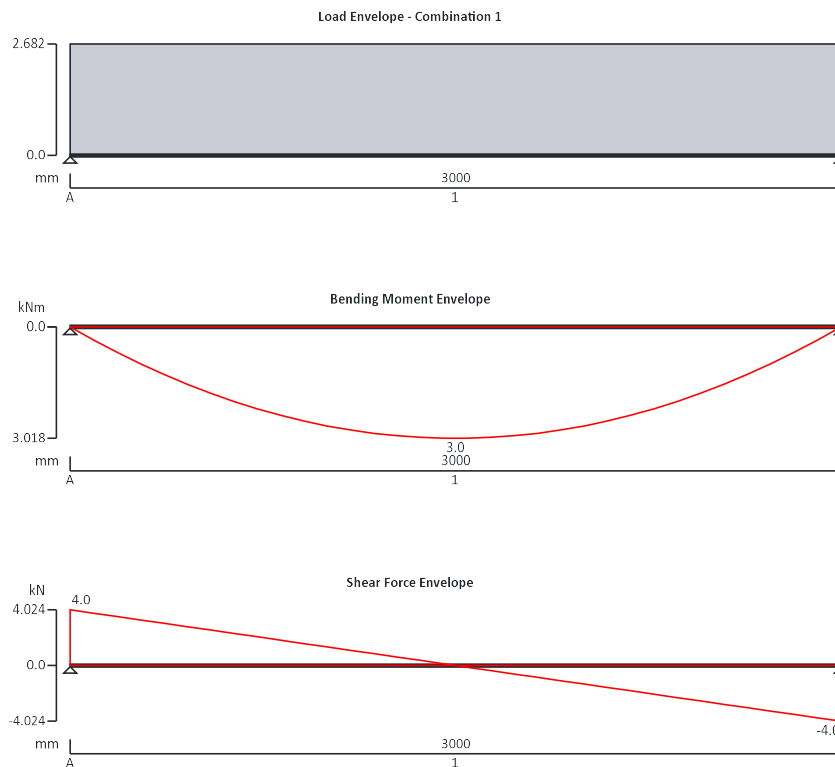
 $w_d = L_{w_ff} \times P_{Roof_d} = 1.63kN/m$

Imposed Load Applied to trimmer;

 $w_{im} = L_{w_ff} \times P_{Roof_im} = 0.90kN/m$

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03



Applied loading

Beam loads

Dead self weight of beam $\times 1$

Dead full UDL 1.700 kN/m

Imposed full UDL 0.900 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.00$

Imposed $\times 1.00$

Span 1

Dead $\times 1.00$

Imposed $\times 1.00$

Support B

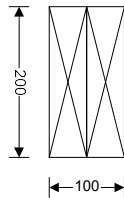
Dead $\times 1.00$

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Imposed $\times 1.00$

Analysis results

Design moment; $M = 3.018 \text{ kNm}$; Design shear; $F = 4.024 \text{ kN}$
Total load on beam; $W_{\text{tot}} = 8.047 \text{ kN}$
Reactions at support A; $R_{A_{\text{max}}} = 4.024 \text{ kN}$; $R_{A_{\text{min}}} = 4.024 \text{ kN}$
Unfactored dead load reaction at support A; $R_{A_{\text{Dead}}} = 2.674 \text{ kN}$
Unfactored imposed load reaction at support A; $R_{A_{\text{Imposed}}} = 1.350 \text{ kN}$
Reactions at support B; $R_{B_{\text{max}}} = 4.024 \text{ kN}$; $R_{B_{\text{min}}} = 4.024 \text{ kN}$
Unfactored dead load reaction at support B; $R_{B_{\text{Dead}}} = 2.674 \text{ kN}$
Unfactored imposed load reaction at support B; $R_{B_{\text{Imposed}}} = 1.350 \text{ kN}$



Timber section details

Breadth of section; $b = 50 \text{ mm}$; Depth of section; $h = 200 \text{ mm}$
Number of sections; $N = 2$; Breadth of beam; $b_b = 100 \text{ mm}$
Timber strength class; **C24**

Member details

Service class of timber; **1**; Load duration; **Long term**
Length of span; $L_{s1} = 3000 \text{ mm}$
Length of bearing; $L_b = 100 \text{ mm}$

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio; **2.00**; Actual depth-to-breadth ratio; **2.00**
PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress; $\sigma_{c_{\text{adm}}} = 2.400 \text{ N/mm}^2$; Applied bearing stress; $\sigma_{c_{\text{a}}} = 0.402 \text{ N/mm}^2$
PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress; $\sigma_{m_{\text{adm}}} = 7.842 \text{ N/mm}^2$; Applied bending stress; $\sigma_{m_{\text{a}}} = 4.527 \text{ N/mm}^2$
PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress; $\tau_{\text{adm}} = 0.710 \text{ N/mm}^2$; Applied shear stress; $\tau_{\text{a}} = 0.302 \text{ N/mm}^2$
PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection; $\delta_{\text{adm}} = 9.000 \text{ mm}$; Total deflection; $\delta_{\text{a}} = 6.296 \text{ mm}$
PASS - Total deflection is less than permissible deflection

Design Summary: Use 2No. 50x200 C24 Timbers to act as stair trimmers.

TIMBER TRIMMER T2 - DESIGN

Design Data:

Design Length; $L_{des} = 4.00\text{m}$
 Loaded Width for floor; $L_{w_ff} = 0.40\text{m}$

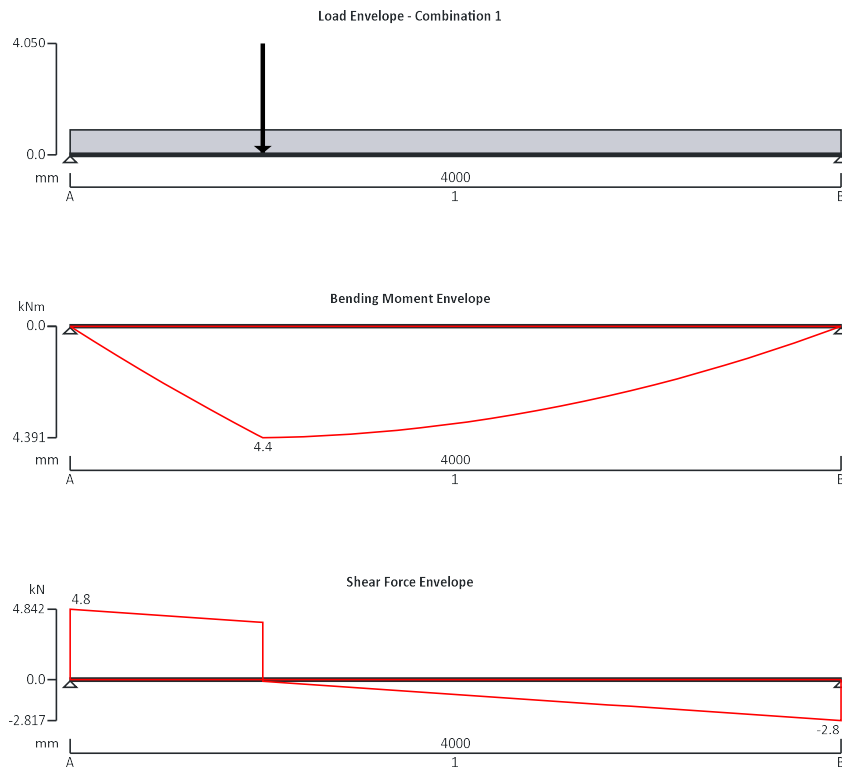
Dead Load Applied to trimmer; $w_d = L_{w_ff} \times P_{Roof_d} = 0.43\text{kN/m}$
 Imposed Load Applied to trimmer; $w_{im} = L_{w_ff} \times P_{Roof_im} = 0.24\text{kN/m}$

Additional Point Load from trimmer T1 at 1000mm:

Dead Reaction from T1; $R_d = 2.7\text{kN}$
 Imposed Reaction from T1; $R_{im} = 1.35\text{kN}$

FLITCH BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03



Applied loading

Beam loads

Dead self weight of beam $\times 1$
 Dead full UDL 0.450 kN/m
 Imposed full UDL 0.250 kN/m
 Dead point load 2.700 kN at 1000 mm

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Imposed point load 1.350 kN at 1000 mm

Load combinations

Load combination 1

Support A

Dead $\times$ 1.00

Imposed $\times$ 1.00

Span 1

Dead $\times$ 1.00

Imposed $\times$ 1.00

Support B

Dead $\times$ 1.00

Imposed $\times$ 1.00

Analysis results

Design moment; $M = 4.391$ kNm; Design shear; $F = 4.842$ kN

Total load on beam; $W_{tot} = 7.660$ kN

Reactions at support A; $R_{A_max} = 4.842$ kN; $R_{A_min} = 4.842$ kN

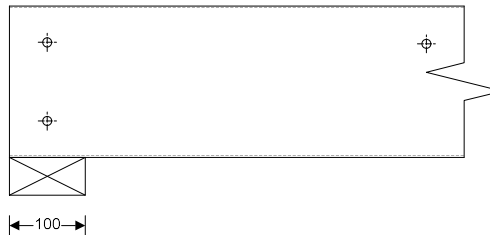
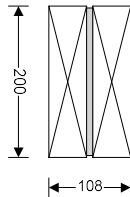
Unfactored dead load reaction at support A; $R_{A_Dead} = 3.330$ kN

Unfactored imposed load reaction at support A; $R_{A_Imposed} = 1.513$ kN

Reactions at support B; $R_{B_max} = 2.817$ kN; $R_{B_min} = 2.817$ kN

Unfactored dead load reaction at support B; $R_{B_Dead} = 1.980$ kN

Unfactored imposed load reaction at support B; $R_{B_Imposed} = 0.837$ kN



Timber section details

Breadth of section; $b = 50$ mm;

Depth of section;

$h = 200$ mm

Number of sections; $N = 2$

Timber strength class; **C24**

Steel section details

Breadth of steel plate; $b_s = 8$ mm;

Depth of steel plate;

$h_s = 195$ mm

Number of steel plates in beam; $N_s = 1$;

Steel stress;

$p_y = 165$ N/mm²

Bolt diameter; $\phi_b = 12$ mm;

Maximum bolt spacing;

$S_{max} = 500$ mm

Member details

Service class of timber; **1**;

Load duration;

Long term

Length of span; $L_{S1} = 4000$ mm

Length of bearing; $L_b = 100$ mm

The beam is part of a load-sharing system consisting of four or more members

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio; **3.00**;

Actual depth-to-breadth ratio; **1.85**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress; $\sigma_{c_adm} = 2.640$ N/mm²;

Applied bearing stress;

$\sigma_{c_a} = 0.484$ N/mm²

PASS - Applied compressive stress is less than permissible compressive stress at bearing

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Bending parallel to grain

Permiss. timber bending stress; $\sigma_{m_adm} = 8.626 \text{ N/mm}^2$; Applied timber bending stress; $\sigma_{m_a} = 2.736 \text{ N/mm}^2$

PASS - Timber bending stress is less than permissible timber bending stress

Permiss. steel bending stress; $p_y = 165.000 \text{ N/mm}^2$; Applied steel bending stress; $\sigma_{m_a_s} = 72.061 \text{ N/mm}^2$

PASS - Steel bending stress is less than permissible steel bending stress

Shear parallel to grain

Permissible shear stress; $\tau_{adm} = 0.781 \text{ N/mm}^2$; Applied shear stress; $\tau_a = 0.151 \text{ N/mm}^2$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection; $\delta_{adm} = 12.000 \text{ mm}$; Total deflection; $\delta_a = 4.287 \text{ mm}$

PASS - Total deflection is less than permissible deflection

Flitch plate bolting requirements

Bolts required at beam end; $N_{be} = 2.000$; Bolts required to beam length; $N_{bl} = 1.701$

- Provide a minimum of 2 No.12 mm diameter bolts at each support
- Provide 12 mm diameter bolts at a maximum of 500 mm centres along the length of the beam

Minimum bolt spacings

Minimum end spacing; $S_{end} = 48 \text{ mm}$; Minimum edge spacing; $S_{edge} = 48 \text{ mm}$

Minimum bolt spacing; $S_{bolt} = 48 \text{ mm}$

Minimum washer diameter; $\phi_w = 36 \text{ mm}$; Minimum washer thickness; $t_w = 3.0 \text{ mm}$

Design Summary: Use 2No. 50 x 200 C24 timber flitch beam with 8 x 195mm Steel plate fixed inbetween.

TIMBER TRIMMER T3 - DESIGN

Design Data:

 Design Length; $L_{des} = 3.00\text{m}$

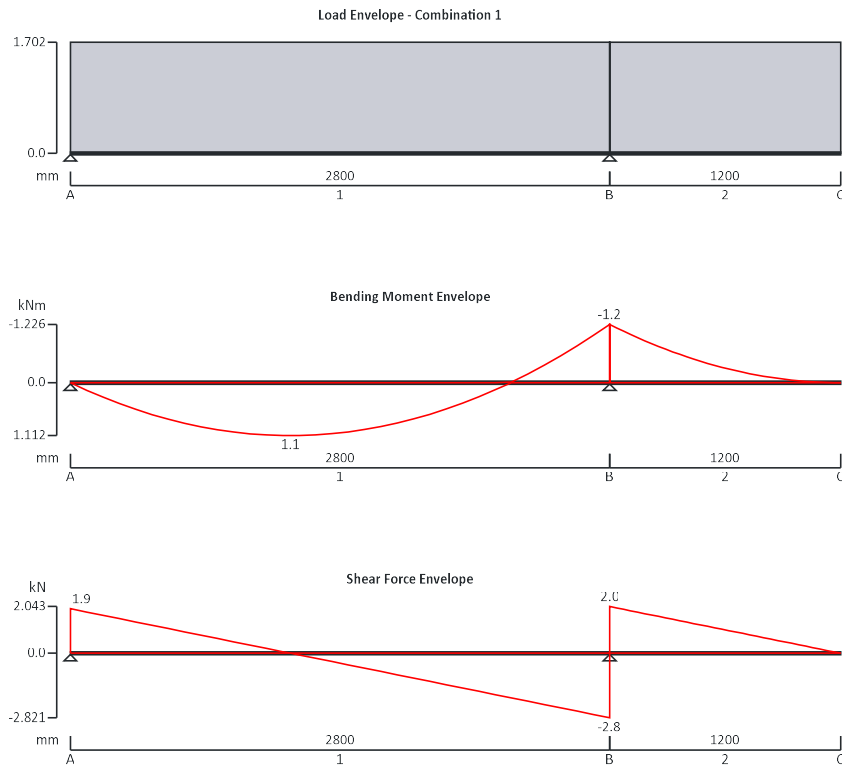
 Loaded Width for roof on span 1 ; $L_w = 0.50\text{m}$

 Dead Load Applied to beam; $w_d = (L_w \times FFloor_d) = 0.25\text{kN/m}$

 Imposed Load Applied to beam; $w_{im} = (L_w \times FFloor_{im}) = 1.25\text{kN/m}$

FLITCH BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03



Applied loading

Beam loads

 Dead self weight of beam $\times 1$

 Dead full UDL 0.250 kN/m

 Imposed full UDL 1.250 kN/m

Load combinations

Load combination 1

Support A

 Dead $\times 1.00$

 Imposed $\times 1.00$

Span 1

 Dead $\times 1.00$

 Imposed $\times 1.00$

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Support B

Dead $\times 1.00$

Imposed $\times 1.00$

Span 2

Dead $\times 1.00$

Imposed $\times 1.00$

Support C

Dead $\times 1.00$

Imposed $\times 1.00$

Analysis results

Design moment; $M = 1.226 \text{ kNm}$; Design shear; $F = 2.821 \text{ kN}$

Total load on beam; $W_{\text{tot}} = 6.810 \text{ kN}$

Reactions at support A; $R_{A_{\text{max}}} = 1.946 \text{ kN}$; $R_{A_{\text{min}}} = 1.946 \text{ kN}$

Unfactored dead load reaction at support A; $R_{A_{\text{Dead}}} = 0.517 \text{ kN}$

Unfactored imposed load reaction at support A; $R_{A_{\text{Imposed}}} = 1.429 \text{ kN}$

Reactions at support B; $R_{B_{\text{max}}} = 4.864 \text{ kN}$; $R_{B_{\text{min}}} = 4.864 \text{ kN}$

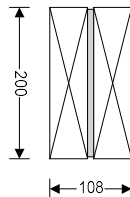
Unfactored dead load reaction at support B; $R_{B_{\text{Dead}}} = 1.293 \text{ kN}$

Unfactored imposed load reaction at support B; $R_{B_{\text{Imposed}}} = 3.571 \text{ kN}$

Reactions at support C; $R_{C_{\text{max}}} = 0.000 \text{ kN}$; $R_{C_{\text{min}}} = 0.000 \text{ kN}$

Unfactored dead load reaction at support C; $R_{C_{\text{Dead}}} = 0.000 \text{ kN}$

Unfactored imposed load reaction at support C; $R_{C_{\text{Imposed}}} = 0.000 \text{ kN}$



Timber section details

Breadth of section; $b = 50 \text{ mm}$; Depth of section; $h = 200 \text{ mm}$

Number of sections; $N = 2$

Timber strength class; **C24**

Steel section details

Breadth of steel plate; $b_s = 8 \text{ mm}$; Depth of steel plate; $h_s = 195 \text{ mm}$

Number of steel plates in beam; $N_s = 1$; Steel stress; $p_y = 165 \text{ N/mm}^2$

Bolt diameter; $\phi_b = 12 \text{ mm}$; Maximum bolt spacing; $S_{\text{max}} = 475 \text{ mm}$

Member details

Service class of timber; **1**; Load duration; **Long term**

Length of span 1; $L_{s1} = 2800 \text{ mm}$

Length of span 2; $L_{s2} = 1200 \text{ mm}$

Length of bearing; $L_b = 100 \text{ mm}$

The beam is part of a load-sharing system consisting of four or more members

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio; **2.00**; Actual depth-to-breadth ratio; **1.85**

PASS - Lateral support is adequate

Check bearing stress

Permissible bearing stress; $\sigma_{c_{\text{adm}}} = 2.904 \text{ N/mm}^2$; Applied bearing stress; $\sigma_{c_a} = 0.486 \text{ N/mm}^2$

PASS - Applied compressive stress is less than permissible compressive stress at bearing

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Bending parallel to grain

Permiss. timber bending stress; $\sigma_{m_adm} = 8.626 \text{ N/mm}^2$; Applied timber bending stress; $\sigma_{m_a} = 0.764 \text{ N/mm}^2$

PASS - Timber bending stress is less than permissible timber bending stress

Permiss. steel bending stress; $p_y = 165.000 \text{ N/mm}^2$; Applied steel bending stress; $\sigma_{m_a_s} = 19.459 \text{ N/mm}^2$

PASS - Steel bending stress is less than permissible steel bending stress

Shear parallel to grain

Permissible shear stress; $\tau_{adm} = 0.781 \text{ N/mm}^2$; Applied shear stress; $\tau_a = 0.088 \text{ N/mm}^2$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Permissible deflection; $\delta_{adm} = 8.400 \text{ mm}$; Total deflection; $\delta_a = 0.544 \text{ mm}$

PASS - Total deflection is less than permissible deflection

Flitch plate bolting requirements

Bolts required at beam end; $N_{be} = 2.000$; Bolts required to beam length; $N_{bl} = 1.463$

- Provide a minimum of 2 No.12 mm diameter bolts at each support
- Provide 12 mm diameter bolts at a maximum of 475 mm centres along the length of the beam

Minimum bolt spacings

Minimum end spacing; $S_{end} = 48 \text{ mm}$; Minimum edge spacing; $S_{edge} = 48 \text{ mm}$

Minimum bolt spacing; $S_{bolt} = 48 \text{ mm}$

Minimum washer diameter; $\phi_w = 36 \text{ mm}$; Minimum washer thickness; $t_w = 3.0 \text{ mm}$

Design Summary: Use 2No. 50 x 200 C24 Timbers with 1No. 8 x 195 Steel plate flitch beam

TIMBER TRIMMER T4 - DESIGN

Design Data:

Design Length; $L_{des} = 1.90\text{m}$
 Loaded Width for floor; $L_w = 1.90\text{m} / 2 = 0.95\text{m}$

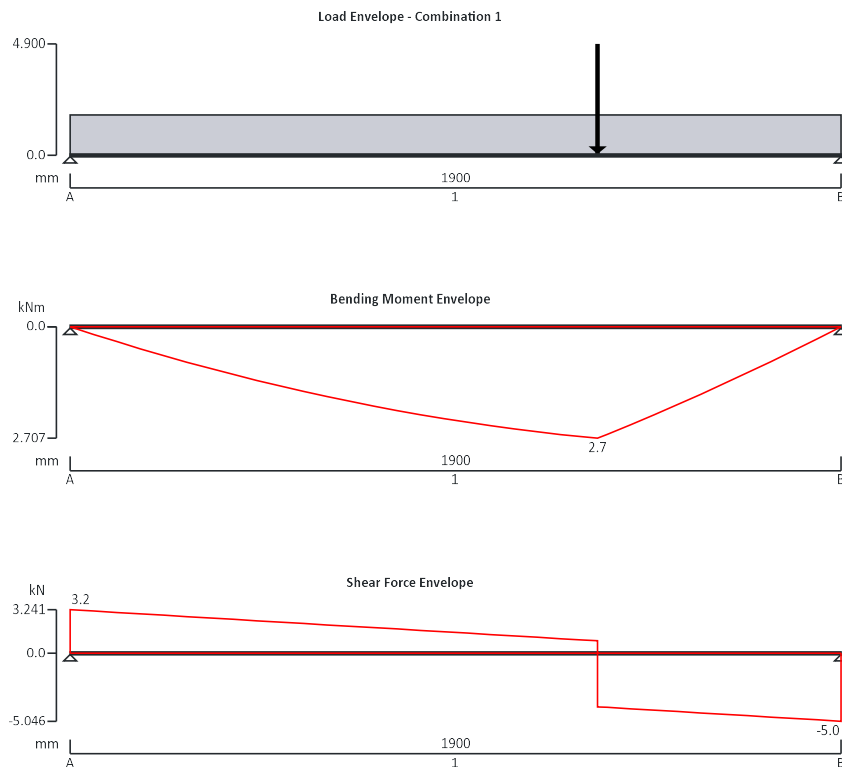
Dead Load Applied to trimmer; $w_d = L_w \times P_{Roof_d} = 1.03\text{kN/m}$
 Imposed Load Applied to trimmer; $w_{im} = L_w \times P_{Roof_im} = 0.57\text{kN/m}$

Additional Point Load applied from T3 @ 1300

Dead Point Load from T3; $R_d = 1.3\text{kN}$
 Imposed Point Load from T3; $R_{im} = 3.6\text{kN}$

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.03



Applied loading

Beam loads

Dead self weight of beam $\times 1$
 Dead full UDL 1.100 kN/m
 Imposed full UDL 0.600 kN/m
 Dead point load 1.300 kN at 1300 mm
 Dead point load 3.600 kN at 1300 mm

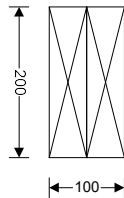
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Load combinations

Load combination 1	Support A	Dead $\times 1.00$ Imposed $\times 1.00$
	Span 1	Dead $\times 1.00$ Imposed $\times 1.00$
	Support B	Dead $\times 1.00$ Imposed $\times 1.00$

Analysis results

Design moment;	$M = 2.707 \text{ kNm};$	Design shear;	$F = 5.046 \text{ kN}$
Total load on beam;	$W_{\text{tot}} = 8.287 \text{ kN}$		
Reactions at support A;	$R_{A_{\text{max}}} = 3.241 \text{ kN};$	$R_{A_{\text{min}}} = 3.241 \text{ kN}$	
Unfactored dead load reaction at support A;	$R_{A_{\text{Dead}}} = 2.671 \text{ kN}$		
Unfactored imposed load reaction at support A;	$R_{A_{\text{Imposed}}} = 0.570 \text{ kN}$		
Reactions at support B;	$R_{B_{\text{max}}} = 5.046 \text{ kN};$	$R_{B_{\text{min}}} = 5.046 \text{ kN}$	
Unfactored dead load reaction at support B;	$R_{B_{\text{Dead}}} = 4.476 \text{ kN}$		
Unfactored imposed load reaction at support B;	$R_{B_{\text{Imposed}}} = 0.570 \text{ kN}$		



Timber section details

Breadth of section;	$b = 50 \text{ mm};$	Depth of section;	$h = 200 \text{ mm}$
Number of sections;	$N = 2;$	Breadth of beam;	$b_b = 100 \text{ mm}$
Timber strength class;	C24		

Member details

Service class of timber;	1;	Load duration;	Long term
Length of span;	$L_{s1} = 1900 \text{ mm}$		
Length of bearing;	$L_b = 100 \text{ mm}$		

Lateral support - cl.2.10.8

Permiss.depth-to-breadth ratio; 2.00;	Actual depth-to-breadth ratio; 2.00
PASS - Lateral support is adequate	

Check bearing stress

Permissible bearing stress;	$\sigma_{c_{\text{adm}}} = 2.400 \text{ N/mm}^2;$	Applied bearing stress;	$\sigma_{c_a} = 0.505 \text{ N/mm}^2$
PASS - Applied compressive stress is less than permissible compressive stress at bearing			

Bending parallel to grain

Permissible bending stress;	$\sigma_{m_{\text{adm}}} = 7.842 \text{ N/mm}^2;$	Applied bending stress;	$\sigma_{m_a} = 4.060 \text{ N/mm}^2$
PASS - Applied bending stress is less than permissible bending stress			

Shear parallel to grain

Permissible shear stress;	$\tau_{\text{adm}} = 0.710 \text{ N/mm}^2;$	Applied shear stress;	$\tau_a = 0.378 \text{ N/mm}^2$
PASS - Applied shear stress is less than permissible shear stress			

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Deflection

Permissible deflection;

$\delta_{adm} = 5.700$ mm;

Total deflection;

$\delta_a = 2.204$ mm

PASS - Total deflection is less than permissible deflection

Design Summary: Use 2No. 50 x 200 Timber beam to act as stair trimmer

BEAM B1 - DESIGN

Design Length; $L_{des} = 6.00\text{m}$
 Loaded Width for roof; $L_{w_rf} = 2.30\text{m} / 2 = 1.15\text{m}$
 Loaded Width for second floor; $L_{w_sf} = 4.60\text{m} / 2 = 2.30\text{m}$
 Loaded Width for first floor; $L_{w_ff} = 4.60\text{m} / 2 = 2.30\text{m}$
 Height of masonry above; $H_{mas} = 5.00\text{m}$

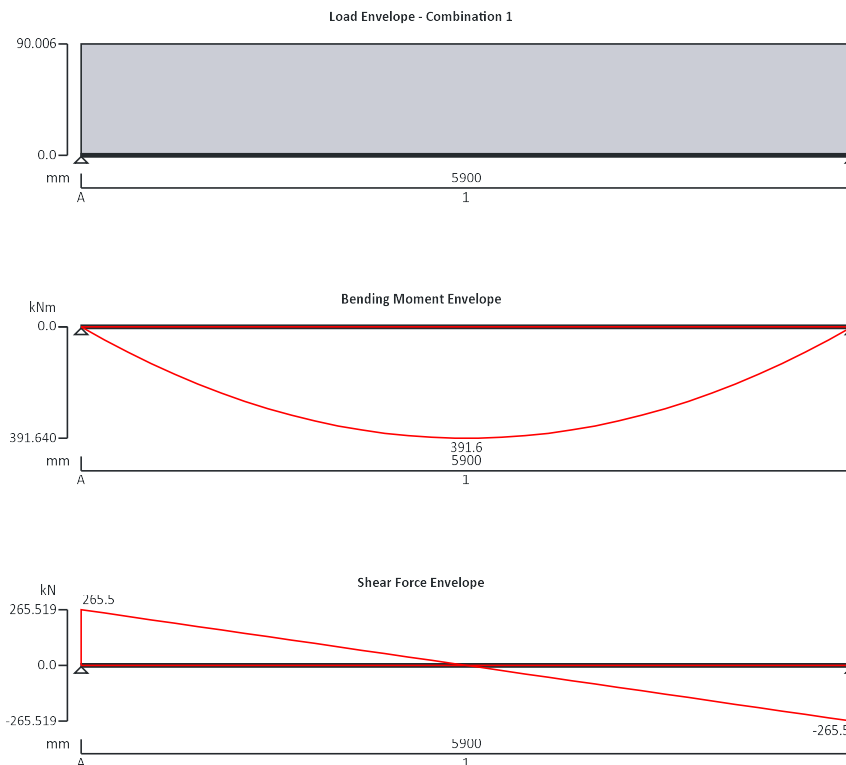
Dead Load Applied to beam;
 $W_d = (L_{w_rf} \times P_{Roof_d}) + (L_{w_sf} \times F_{Floor_d}) + (L_{w_ff} \times F_{Floor_d}) + (H_{mas} \times \text{Wall_d}) = 54.55\text{kN/m}$

Imposed Load Applied to beam;
 $W_{im} = (L_{w_rf} \times P_{Roof_im}) + (L_{w_sf} \times F_{Floor_im}) + (L_{w_ff} \times F_{Floor_im}) = 7.59\text{kN/m}$

STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.08



Support conditions

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

Applied loading

Beam loads

Dead self weight of beam $\times 1$
Dead full UDL 54.6 kN/m
Imposed full UDL 7.7 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$
Imposed $\times 1.60$
Dead $\times 1.40$
Imposed $\times 1.60$

Support B

Dead $\times 1.40$
Imposed $\times 1.60$

Analysis results

Maximum moment;

$M_{max} = 391.6$ kNm;

$M_{min} = 0$ kNm

Maximum shear;

$V_{max} = 265.5$ kN;

$V_{min} = -265.5$ kN

Deflection;

$\delta_{max} = 19.9$ mm;

$\delta_{min} = 0$ mm

Maximum reaction at support A;

$R_{A_{max}} = 265.5$ kN;

$R_{A_{min}} = 265.5$ kN

Unfactored dead load reaction at support A;

$R_{A_{Dead}} = 163.7$ kN

Unfactored imposed load reaction at support A;

$R_{A_{Imposed}} = 22.7$ kN

Maximum reaction at support B;

$R_{B_{max}} = 265.5$ kN;

$R_{B_{min}} = 265.5$ kN

Unfactored dead load reaction at support B;

$R_{B_{Dead}} = 163.7$ kN

Unfactored imposed load reaction at support B;

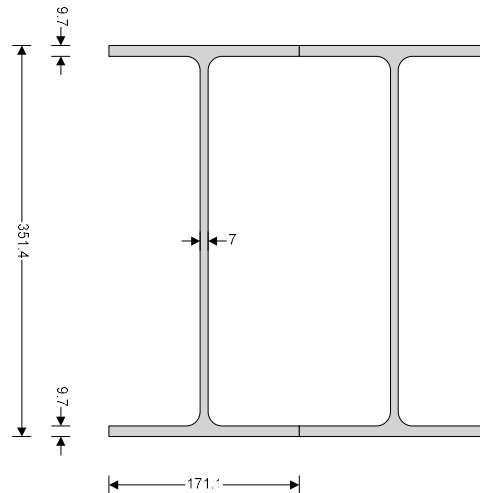
$R_{B_{Imposed}} = 22.7$ kN

Section details

Section type;

2 x UB 356x171x45 (British Steel Section Range 2022 (BS4-1));
S355

Steel grade;



Classification of cross sections - Section 3.5

Tensile strain coefficient; $\epsilon = 0.88$;

Section classification;

Semi-compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = 265.5$ kN;

Design shear resistance; $P_v = 1047.9$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = 391.6$ kNm;

Moment capacity low shear; $M_c = 555.3$ kNm

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PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$\delta_{lim} = 23.6$ mm;

Maximum deflection;

$\delta = 19.94$ mm

PASS - Maximum deflection does not exceed deflection limit

Design Summary: Use 2No. 356x171x45 UB Beams bolted together at 600mm centres.

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COLUMN C1 - DESIGN

Design Data:

Height of Column;	$H_{col} = 2.40\text{m}$
Point Load Applied to column from beam B1;	
Unfactored Dead Reaction from beam B1;	$R_d = 163.7\text{kN}$
Unfactored Imposed Reaction from B1;	$R_{im} = 36\text{kN}$
Maximum Reaction from beam B1;	$R_{max} = 286.8\text{kN}$
Moment acting at column head;	$M_{y,Ed} = 43.1$

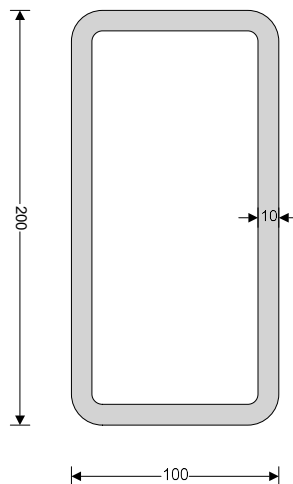
STEEL MEMBER DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.08

Section details

Section type; **RHS 200x100x10.0 (Tata Steel Celsius (Gr355 Gr420));** Steel grade; **S355**



Classification of cross sections - Section 3.5

Tensile strain coefficient; $\epsilon = 0.88$; Section classification; **Semi-compact**

Moment capacity - Section 4.2.5

Design bending moment; $M = 43.1 \text{ kNm}$; Moment capacity low shear; $M_c = 73.2 \text{ kNm}$

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force; $F_c = 286.8 \text{ kN}$; Compression resistance; $P_{cx} = 1829.8 \text{ kN}$

PASS - Compression resistance exceeds design compression force

Design compression force; $F_c = 286.8 \text{ kN}$; Compression resistance; $P_{cy} = 1496.1 \text{ kN}$

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comp.and bending check; $F_c / (A \times p_y) + M / M_c = 0.736$

PASS - Combined bending and compression check is satisfied

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Member buckling resistance - cl.4.8.3.3.3

Buckling resistance checks; $F_c / P_{cy} + m_y \times M / M_c \times (1 + 0.5 \times F_c / P_{cy}) = \mathbf{0.837}$

$F_c / P_{cx} + 0.5 \times m_{yx} \times M_y / M_{cy} = \mathbf{0.451}$

PASS - Member buckling resistance checks are satisfied



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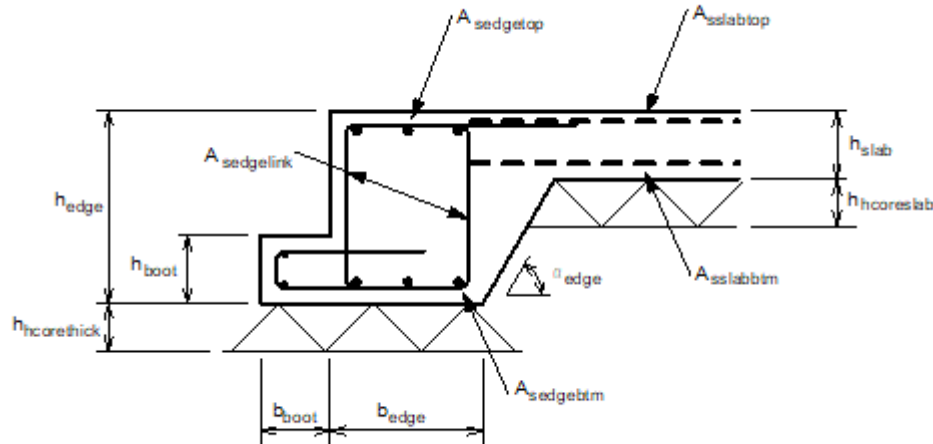
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RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)

Tedds calculation version 1.0.13



Soil and raft definition

Soil definition

Allowable bearing pressure
Number of types of soil forming sub-soil
Soil density
Depth of hardcore beneath slab
Depth of hardcore beneath thickenings
Density of hardcore
Basic assumed diameter of local depression
Diameter under slab modified for hardcore
Diameter under thickenings modified for hardcore

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

Two or more types

Firm to loose

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

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

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

$$\phi_{depbasic} = 3500 \text{ mm}$$

$$\phi_{dep_slab} = \phi_{depbasic} - h_{core_slab} = 3350 \text{ mm}$$

$$\phi_{dep_thick} = \phi_{depbasic} - h_{core_thick} = 3350 \text{ mm}$$

Raft slab definition

Max dimension/max dimension between joints
Slab thickness
Concrete strength
Poissons ratio of concrete
Slab mesh reinforcement strength
Partial safety factor for steel reinforcement
From C&CA document 'Concrete ground floors' Table 5

$$l_{max} = 6.000 \text{ m}$$

$$h_{slab} = 150 \text{ mm}$$

$$f_{cu} = 35 \text{ N/mm}^2$$

$$\nu = 0.2$$

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

$$\gamma_s = 1.15$$

Minimum mesh required in top for shrinkage

A142

Actual mesh provided in top

$$\mathbf{A252} \text{ (} A_{ssl_top} = 252 \text{ mm}^2/\text{m} \text{)}$$

Mesh provided in bottom

$$\mathbf{A252} \text{ (} A_{ssl_bot} = 252 \text{ mm}^2/\text{m} \text{)}$$

Top mesh bar diameter

$$\phi_{slab_top} = 8 \text{ mm}$$

Bottom mesh bar diameter

$$\phi_{slab_bot} = 8 \text{ mm}$$

Cover to top reinforcement

$$C_{top} = 20 \text{ mm}$$

Cover to bottom reinforcement

$$C_{bot} = 35 \text{ mm}$$

Average effective depth of top reinforcement

$$d_{tslab} = h_{slab} - C_{top} - \phi_{slab_top} = 122 \text{ mm}$$

Average effective depth of bottom reinforcement

$$d_{bslab} = h_{slab} - C_{bot} - \phi_{slab_bot} = 107 \text{ mm}$$

Overall average effective depth

$$d_{slab} = (d_{tslab} + d_{bslab})/2 = 114 \text{ mm}$$

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Minimum effective depth of top reinforcement

$$d_{tslabmin} = d_{tslabav} - \phi_{slabtop}/2 = \mathbf{118 \text{ mm}}$$

Minimum effective depth of bottom reinforcement

$$d_{bslabmin} = d_{bslabav} - \phi_{slabbtm}/2 = \mathbf{103 \text{ mm}}$$

Edge beam definition

Overall depth

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

Width

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

Depth of boot

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

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

$$\mathbf{3 \text{ H16 bars } (A_{sedge\text{top}} = 603 \text{ mm}^2)}$$

Reinforcement provided in bottom

$$\mathbf{3 \text{ H16 bars } (A_{sedge\text{btm}} = 603 \text{ mm}^2)}$$

Link reinforcement provided

$$\mathbf{2 \text{ H10 legs at 250 ctrs } (A_{sv}/s_v = 0.628 \text{ mm})}$$

Bottom cover to links

$$C_{beam} = \mathbf{35 \text{ mm}}$$

Effective depth of top reinforcement

$$d_{edge\text{top}} = h_{edge} - C_{top} - \phi_{slabtop} - \phi_{edgelink} - \phi_{edge\text{top}}/2 = \mathbf{404 \text{ mm}}$$

Effective depth of bottom reinforcement

$$d_{edge\text{btm}} = h_{edge} - C_{beam} - \phi_{edgelink} - \phi_{edge\text{btm}}/2 = \mathbf{397 \text{ mm}}$$

Boot main reinforcement

$$\mathbf{H16 \text{ bars at 300 ctrs } (A_{sboot} = 670 \text{ mm}^2/\text{m})}$$

Effective depth of boot reinforcement

$$d_{boot} = h_{boot} - C_{beam} - \phi_{boot}/2 = \mathbf{182 \text{ mm}}$$

Internal slab design checks

Basic loading

Slab self weight

$$W_{slab} = 24 \text{ kN/m}^3 \times h_{slab} = \mathbf{3.6 \text{ kN/m}^2}$$

Hardcore

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

Applied loading

Uniformly distributed dead load

$$W_{Dudl} = \mathbf{0.5 \text{ kN/m}^2}$$

Uniformly distributed live load

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

Internal slab bearing pressure check

Total uniform load at formation level

$$W_{udl} = W_{slab} + W_{hcoreslab} + W_{Dudl} + W_{Ludl} = \mathbf{8.6 \text{ kN/m}^2}$$

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

$$l_{slab} = \phi_{depslab} + d_{tslabav} = \mathbf{3472 \text{ mm}}$$

Ultimate self weight udl

$$W_{swult} = 1.4 \times W_{slab} = \mathbf{5.0 \text{ kN/m}^2}$$

Self weight moment at centre

$$M_{csw} = W_{swult} \times l_{slab}^2 \times (1 + \nu) / 64 = \mathbf{1.1 \text{ kNm/m}}$$

Self weight moment at edge

$$M_{esw} = W_{swult} \times l_{slab}^2 / 32 = \mathbf{1.9 \text{ kNm/m}}$$

Self weight shear force at edge

$$V_{sw} = W_{swult} \times l_{slab} / 4 = \mathbf{4.4 \text{ kN/m}}$$

Moments due to applied uniformly distributed loads

Ultimate applied udl

$$W_{dulult} = 1.4 \times W_{Dudl} + 1.6 \times W_{Ludl} = \mathbf{3.1 \text{ kN/m}^2}$$

Moment at centre

$$M_{cudl} = W_{dulult} \times l_{slab}^2 \times (1 + \nu) / 64 = \mathbf{0.7 \text{ kNm/m}}$$

Moment at edge

$$M_{eudl} = W_{dulult} \times l_{slab}^2 / 32 = \mathbf{1.2 \text{ kNm/m}}$$

Shear force at edge

$$V_{udl} = W_{dulult} \times l_{slab} / 4 = \mathbf{2.7 \text{ kN/m}}$$

Resultant moments and shears

Total moment at edge

$$M_{\Sigma e} = \mathbf{3.1 \text{ kNm/m}}$$

Total moment at centre

$$M_{\Sigma c} = \mathbf{1.8 \text{ kNm/m}}$$

Total shear force

$$V_{\Sigma} = \mathbf{7.1 \text{ kN/m}}$$

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Reinforcement required in top

K factor	$K_{slabtop} = M_{\Sigma e} / (f_{cu} \times d_{slabav}^2) = 0.006$
Lever arm	$Z_{slabtop} = d_{slabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtop}/0.9)}) = 115.9 \text{ mm}$
Area of steel required for bending	$A_{slabtopbend} = M_{\Sigma e} / ((1.0/\gamma_s) \times f_{yslab} \times Z_{slabtop}) = 61 \text{ mm}^2/\text{m}$
Minimum area of steel required	$A_{slabmin} = 0.0013 \times h_{slab} = 195 \text{ mm}^2/\text{m}$
Area of steel required	$A_{slabtopreq} = \max(A_{slabtopbend}, A_{slabmin}) = 195 \text{ mm}^2/\text{m}$
PASS - $A_{slabtopreq} \leq A_{slabtop}$ - Area of reinforcement provided in top to span local depressions is adequate	

Reinforcement required in bottom

K factor	$K_{slabbtm} = M_{\Sigma c} / (f_{cu} \times d_{slabav}^2) = 0.005$
Lever arm	$Z_{slabbtm} = d_{slabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbtm}/0.9)}) = 101.6 \text{ mm}$
Area of steel required for bending	$A_{slabbtmbend} = M_{\Sigma c} / ((1.0/\gamma_s) \times f_{yslab} \times Z_{slabbtm}) = 42 \text{ mm}^2/\text{m}$
Area of steel required	$A_{slabbtmreq} = \max(A_{slabbtmbend}, A_{slabmin}) = 195 \text{ mm}^2/\text{m}$
PASS - $A_{slabbtmreq} \leq A_{slabbtm}$ - Area of reinforcement provided in bottom to span local depressions is adequate	

Shear check

Applied shear stress	$v = V_{\Sigma} / d_{slabmin} = 0.060 \text{ N/mm}^2$
Tension steel ratio	$\rho = 100 \times A_{slabtop} / d_{slabmin} = 0.214$
From BS8110-1:1997 - Table 3.8	
Design concrete shear strength	$v_c = 0.573 \text{ N/mm}^2$
PASS - $v \leq v_c$ - Shear capacity of the slab is adequate	

Internal slab deflection check

Basic allowable span to depth ratio	$Ratio_{basic} = 26.0$
Moment factor	$M_{factor} = M_{\Sigma c} / d_{slabav}^2 = 0.161 \text{ N/mm}^2$
Steel service stress	$f_s = 2/3 \times f_{yslab} \times A_{slabbtmbend} / A_{slabbtm} = 55.066 \text{ N/mm}^2$
Modification factor	$MF_{slab} = \min(2.0, 0.55 + [(477 \text{ N/mm}^2 - f_s) / (120 \times (0.9 \text{ N/mm}^2 + M_{factor}))])$ $MF_{slab} = 2.000$
Modified allowable span to depth ratio	$Ratio_{allow} = Ratio_{basic} \times MF_{slab} = 52.000$
Actual span to depth ratio	$Ratio_{actual} = l_{slab} / d_{slabav} = 32.449$
PASS - $Ratio_{actual} \leq Ratio_{allow}$ - Slab span to depth ratio is adequate	

Edge beam design checks

Basic loading

Hardcore	$W_{hcorethick} = \gamma_{hcore} \times h_{hcorethick} = 3.0 \text{ kN/m}^2$
Edge beam	
Rectangular beam element	$W_{beam} = 24 \text{ kN/m}^3 \times h_{edge} \times b_{edge} = 4.3 \text{ kN/m}$
Boot element	$W_{boot} = 24 \text{ kN/m}^3 \times h_{boot} \times b_{boot} = 1.1 \text{ kN/m}$
Chamfer element	$W_{chamfer} = 24 \text{ kN/m}^3 \times (h_{edge} - h_{slab})^2 / (2 \times \tan(\alpha_{edge})) = 0.6 \text{ kN/m}$
Slab element	$W_{slabelmt} = 24 \text{ kN/m}^3 \times h_{slab} \times (h_{edge} - h_{slab}) / \tan(\alpha_{edge}) = 0.6 \text{ kN/m}$
Edge beam self weight	$W_{edge} = W_{beam} + W_{boot} + W_{chamfer} + W_{slabelmt} = 6.6 \text{ kN/m}$

Edge load number 1

Load type	Longitudinal line load
Dead load	$W_{Dedge1} = 15.0 \text{ kN/m}$
Live load	$W_{Ledge1} = 1.0 \text{ kN/m}$
Ultimate load	$W_{ultedge1} = 1.4 \times W_{Dedge1} + 1.6 \times W_{Ledge1} = 22.6 \text{ kN/m}$
Longitudinal line load width	$b_{edge1} = 325 \text{ mm}$
Centroid of load from outside face of raft	$X_{edge1} = 0 \text{ mm}$

Edge load number 2

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Load type

Dead load

Live load

Ultimate load

Transverse line load width

Transverse line load

$$W_{Dedge2} = 15.0 \text{ kN/m}$$

$$W_{Ledge2} = 1.0 \text{ kN/m}$$

$$W_{ultedge2} = 1.4 \times W_{Dedge2} + 1.6 \times W_{Ledge2} = 22.6 \text{ kN/m}$$

$$b_{edge2} = 325 \text{ mm}$$

Edge beam bearing pressure check

Effective bearing width of edge beam

$$b_{bearing} = b_{edge} + b_{boot} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}) = 773 \text{ mm}$$

Total uniform load at formation level

$$W_{udledge} = W_{Dudl} + W_{Ludl} + W_{edge}/b_{bearing} + W_{hcorethick} = 13.6 \text{ kN/m}^2$$

Bearing pressure due to transverse line loads

Total dead transverse line load

$$W_{Dtrans} = 15.0 \text{ kN/m}$$

Total live transverse line load

$$W_{Ltrans} = 1.0 \text{ kN/m}$$

Total ultimate transverse line load

$$W_{ulttrans} = 22.6 \text{ kN/m}$$

Minimum width of transverse line loads

$$b_{trans} = 325 \text{ mm}$$

Length of trans line load applied to edge beam

$$l_{transapp} = b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}) = 573 \text{ mm}$$

Total ult trans line load applied to edge beam

$$W_{ulttrans} = W_{ulttrans} \times l_{transapp} = 13.0 \text{ kN}$$

Approx moment capacity of bottom steel

$$M_{edgebtm} = (1.0/\gamma_s) \times f_y \times 0.9 \times d_{edgebtm} \times A_{sedgebtm} = 93.7 \text{ kNm}$$

Max allow dispersal based on moment capacity

$$p_{edgemom} = [2 \times M_{edgebtm} + \sqrt{(4 \times M_{edgebtm}^2 + 2 \times W_{ulttrans} \times M_{edgebtm} \times b_{trans})}] / W_{ulttrans}$$

$$p_{edgemom} = 29095 \text{ mm}$$

Limiting max dispersal to say 5 x beam depth

$$p_{edge} = \min(p_{edgemom}, 5 \times h_{edge}) = 2250 \text{ mm}$$

Total dispersal width of transverse line loads

$$l_{trans} = 2 \times p_{edge} + b_{trans} = 4825 \text{ mm}$$

Bearing pressure due to trans line loads

$$q_{trans} = (W_{Dtrans} + W_{Ltrans}) \times l_{transapp} / (l_{trans} \times b_{bearing}) = 2.5 \text{ kN/m}^2$$

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

Load x distance for edge load 1

$$\text{Moment}_1 = W_{ultedge1} \times X_{edge1} = 0.0 \text{ kN}$$

Sum of ultimate longitud'l and equivalent line loads $\Sigma UDL = 22.6 \text{ kN/m}$

Sum of load x distances

$$\Sigma \text{Moment} = 0.0 \text{ kN}$$

Centroid of loads

$$X_{bar} = \Sigma \text{Moment} / \Sigma UDL = 0 \text{ mm}$$

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

Sum of unfactored longitud'l and eff'tive line loads $\Sigma UDLsls = 16.0 \text{ kN/m}$

Allowable bearing width

$$b_{allow} = 2 \times X_{bar} + 2 \times h_{hcoreslab} \times \tan(30) = 173 \text{ mm}$$

Bearing pressure due to line/point loads

$$q_{linepoint} = \Sigma UDLsls / b_{allow} = 92.4 \text{ kN/m}^2$$

Total applied bearing pressure

$$q_{edge} = q_{linepoint} + q_{trans} + W_{udledge} = 108.4 \text{ kN/m}^2$$

$q_{edge} > q_{allow}$ - 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

$$b_{req} = \Sigma UDLsls / (q_{allow} - q_{trans} - W_{udledge}) = 471 \text{ mm}$$

Effective bearing width at u/s of slab

$$b_{reqeff} = b_{req} - 2 \times h_{hcoreslab} \times \tan(30) = 298 \text{ mm}$$

Load/reaction eccentricity

$$e = b_{reqeff} / 2 - X_{bar} = 149 \text{ mm}$$

Ultimate moment to be resisted by slab

$$M_{ecc} = \Sigma UDL \times e = 3.4 \text{ kNm/m}$$

From slab bending check

Moment due to depression under slab (hogging)

$$M_{\Sigma e} = 3.1 \text{ kNm/m}$$

Total moment to be resisted by slab top steel

$$M_{slabtop} = M_{ecc} + M_{\Sigma e} = 6.4 \text{ kNm/m}$$

K factor

$$K_{slab} = M_{slabtop} / (f_{cu} \times d_{slabmin}^2) = 0.013$$

Lever arm

$$Z_{slab} = d_{slabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slab}/0.9)}) = 112 \text{ mm}$$

Area of steel required

$$A_{slabreq} = M_{slabtop} / ((1.0/\gamma_s) \times f_y \times Z_{slab}) = 132 \text{ mm}^2/\text{m}$$

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

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

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Library item - B pressure with moment transfer

Edge beam bending check

Divider for moments due to udl's $\beta_{udl} = 10.0$

Divider for moments due to point loads $\beta_{point} = 6.0$

Applied bending moments

Span of edge beam $l_{edge} = \phi_{depth} + d_{edgetop} = 3754 \text{ mm}$

Ultimate self weight udl $W_{edgeult} = 1.4 \times W_{edge} = 9.3 \text{ kN/m}$

Ultimate slab udl (approx) $W_{edgeslab} = \max(0 \text{ kN/m}, 1.4 \times W_{slab} \times ((\phi_{depth}/2 \times 3/4) - (b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}))))$
 $W_{edgeslab} = 3.4 \text{ kN/m}$

Self weight and slab bending moment $M_{edgesw} = (W_{edgeult} + W_{edgeslab}) \times l_{edge}^2 / \beta_{udl} = 18.0 \text{ kNm}$

Self weight shear force $V_{edgesw} = (W_{edgeult} + W_{edgeslab}) \times l_{edge} / 2 = 23.9 \text{ kN}$

Moments due to applied uniformly distributed loads

Ultimate udl (approx) $W_{edgeudl} = W_{udlult} \times \phi_{depth} / 2 \times 3/4 = 3.9 \text{ kN/m}$

Bending moment $M_{edgeudl} = W_{edgeudl} \times l_{edge}^2 / \beta_{udl} = 5.5 \text{ kNm}$

Shear force $V_{edgeudl} = W_{edgeudl} \times l_{edge} / 2 = 7.3 \text{ kN}$

Moment and shear due to load number 1

Bending moment $M_{edge1} = W_{ultedge1} \times l_{edge}^2 / \beta_{udl} = 31.8 \text{ kNm}$

Shear force $V_{edge1} = W_{ultedge1} \times l_{edge} / 2 = 42.4 \text{ kN}$

Moment and shear due to load number 2

Ultimate point load $W_{edge2} = W_{ultedge2} \times \phi_{depth} / 2 \times 3/4 = 28.4 \text{ kN}$

Bending moment $M_{edge2} = W_{edge2} \times l_{edge} / \beta_{point} = 17.8 \text{ kNm}$

Shear force $V_{edge2} = W_{edge2} = 28.4 \text{ kN}$

Resultant moments and shears

Total moment (hogging and sagging) $M_{\Sigma edge} = 73.1 \text{ kNm}$

Maximum shear force $V_{\Sigma edge} = 102.1 \text{ kN}$

Reinforcement required in top

Width of section in compression zone $b_{edgetop} = b_{edge} + b_{boot} = 600 \text{ mm}$

Average web width $b_w = b_{edge} + (h_{edge} / \tan(\alpha_{edge})) / 2 = 530 \text{ mm}$

K factor $K_{edgetop} = M_{\Sigma edge} / (f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = 0.021$

Lever arm $z_{edgetop} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgetop}/0.9)}) = 384 \text{ mm}$

Area of steel required for bending $A_{sedgetopbend} = M_{\Sigma edge} / ((1.0/\gamma_s) \times f_y \times z_{edgetop}) = 438 \text{ mm}^2$

Minimum area of steel required $A_{sedgetopmin} = 0.0013 \times 1.0 \times b_w \times h_{edge} = 310 \text{ mm}^2$

Area of steel required $A_{sedgetopreq} = \max(A_{sedgetopbend}, A_{sedgetopmin}) = 438 \text{ mm}^2$

PASS - $A_{sedgetopreq} \leq A_{sedgetop}$ - Area of reinforcement provided in top of edge beams is adequate

Reinforcement required in bottom

Width of section in compression zone $b_{edgebtm} = b_{edge} + (h_{edge} - h_{slab}) / \tan(\alpha_{edge}) + 0.1 \times l_{edge} = 949 \text{ mm}$

K factor $K_{edgebtm} = M_{\Sigma edge} / (f_{cu} \times b_{edgebtm} \times d_{edgebtm}^2) = 0.014$

Lever arm $z_{edgebtm} = d_{edgebtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgebtm}/0.9)}) = 377 \text{ mm}$

Area of steel required for bending $A_{sedgetmbend} = M_{\Sigma edge} / ((1.0/\gamma_s) \times f_y \times z_{edgebtm}) = 446 \text{ mm}^2$

Minimum area of steel required $A_{sedgetbmin} = 0.0013 \times 1.0 \times b_w \times h_{edge} = 310 \text{ mm}^2$

Area of steel required $A_{sedgetbreq} = \max(A_{sedgetmbend}, A_{sedgetbmin}) = 446 \text{ mm}^2$

PASS - $A_{sedgetbreq} \leq A_{sedgetb}$ - Area of reinforcement provided in bottom of edge beams is adequate

Edge beam shear check

Applied shear stress $V_{edge} = V_{\Sigma edge} / (b_w \times d_{edgetop}) = 0.477 \text{ N/mm}^2$

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Tension steel ratio

$$\rho_{edge} = 100 \times A_{s_{edge}} / (b_w \times d_{edge}) = \mathbf{0.282}$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$V_{edge} = \mathbf{0.464 \text{ N/mm}^2}$$

$$V_{edge} \leq V_{edge} + 0.4 \text{ N/mm}^2 - \text{Therefore minimum links required}$$

Link area to spacing ratio required

$$A_{sv_upon_S_{vreqedge}} = 0.4 \text{ N/mm}^2 \times b_w / ((1.0/\gamma_s) \times f_{ys}) = \mathbf{0.488 \text{ mm}}$$

Link area to spacing ratio provided

$$A_{sv_upon_S_{vprovedge}} = N_{edge} \times \pi \times \phi_{edge}^2 / (4 \times S_{vedge}) = \mathbf{0.628 \text{ mm}}$$

$$PASS - A_{sv_upon_S_{vreqedge}} \leq A_{sv_upon_S_{vprovedge}} - \text{Shear reinforcement provided in edge beams is adequate}$$

Boot design check

Effective cantilever span

$$l_{boot} = b_{boot} + d_{boot}/2 = \mathbf{291 \text{ mm}}$$

Approximate ultimate bearing pressure

$$q_{ult} = 1.55 \times q_{allow} = \mathbf{77.5 \text{ kN/m}^2}$$

Cantilever moment

$$M_{boot} = q_{ult} \times l_{boot}^2 / 2 = \mathbf{3.3 \text{ kNm/m}}$$

Shear force

$$V_{boot} = q_{ult} \times l_{boot} = \mathbf{22.6 \text{ kN/m}}$$

K factor

$$K_{boot} = M_{boot} / (f_{cu} \times d_{boot}^2) = \mathbf{0.003}$$

Lever arm

$$Z_{boot} = d_{boot} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{boot}/0.9)}) = \mathbf{173 \text{ mm}}$$

Area of reinforcement required

$$A_{sbootreq} = M_{boot} / ((1.0/\gamma_s) \times f_{yboot} \times Z_{boot}) = \mathbf{44 \text{ mm}^2/\text{m}}$$

$$PASS - A_{sbootreq} \leq A_{sboot} - \text{Area of reinforcement provided in boot is adequate for bending}$$

Applied shear stress

$$v_{boot} = V_{boot} / d_{boot} = \mathbf{0.124 \text{ N/mm}^2}$$

Tension steel ratio

$$\rho_{boot} = 100 \times A_{sboot} / d_{boot} = \mathbf{0.368}$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$V_{cboot} = \mathbf{0.617 \text{ N/mm}^2}$$

$$PASS - v_{boot} \leq V_{cboot} - \text{Shear capacity of the boot is adequate}$$

Corner design checks

Basic loading

Corner bearing pressure check

Total uniform load at formation level

$$W_{udlcorner} = W_{Dudl} + W_{Ludl} + W_{edge} / d_{bearing} + W_{corethick} = \mathbf{13.6 \text{ kN/m}^2}$$

$$PASS - W_{udlcorner} \leq q_{allow} - \text{Applied bearing pressure is less than allowable}$$

Corner beam bending check

Cantilever span of edge beam

$$l_{corner} = \phi_{depth} / \sqrt{2} + d_{edge} / 2 = \mathbf{2571 \text{ mm}}$$

Moment and shear due to self weight

Ultimate self weight udl

$$W_{edgeult} = 1.4 \times W_{edge} = \mathbf{9.3 \text{ kN/m}}$$

Average ultimate slab udl (approx)

$$W_{cornerslab} = \max(0 \text{ kN/m}, 1.4 \times W_{slab} \times (\phi_{depth} / (\sqrt{2} \times 2) - (b_{edge} + (h_{edge} - h_{slab}) / \tan(\alpha_{edge}))))$$

$$W_{cornerslab} = \mathbf{3.1 \text{ kN/m}}$$

Self weight and slab bending moment

$$M_{cornersw} = (W_{edgeult} + W_{cornerslab}) \times l_{corner}^2 / 2 = \mathbf{40.9 \text{ kNm}}$$

Self weight and slab shear force

$$V_{cornersw} = (W_{edgeult} + W_{cornerslab}) \times l_{corner} = \mathbf{31.8 \text{ kN}}$$

Moment and shear due to udls

Maximum ultimate udl

$$W_{cornerudl} = ((1.4 \times W_{Dudl}) + (1.6 \times W_{Ludl})) \times \phi_{depth} / \sqrt{2} = \mathbf{7.3 \text{ kN/m}}$$

Bending moment

$$M_{cornerudl} = W_{cornerudl} \times l_{corner}^2 / 6 = \mathbf{8.1 \text{ kNm}}$$

Shear force

$$V_{cornerudl} = W_{cornerudl} \times l_{corner} / 2 = \mathbf{9.4 \text{ kN}}$$

Resultant moments and shears

Total design moment

$$M_{\Sigma corner} = M_{cornersw} + M_{cornerudl} = \mathbf{49.0 \text{ kNm}}$$

Total design shear force

$$V_{\Sigma corner} = V_{cornersw} + V_{cornerudl} = \mathbf{41.3 \text{ kN}}$$

Reinforcement required in top of edge beam

K factor

$$K_{corner} = M_{\Sigma corner} / (f_{cu} \times b_{edge} \times d_{edge}^2) = \mathbf{0.014}$$

Lever arm

$$Z_{corner} = d_{edge} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{corner}/0.9)}) = \mathbf{384 \text{ mm}}$$

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Area of steel required for bending

$$A_{\text{scornerbend}} = M_{\Sigma\text{corner}} / ((1.0/\gamma_s) \times f_y \times Z_{\text{corner}}) = \mathbf{294 \text{ mm}^2}$$

Minimum area of steel required

$$A_{\text{scornermin}} = A_{\text{sedgetopmin}} = \mathbf{310 \text{ mm}^2}$$

Area of steel required

$$A_{\text{scorner}} = \max(A_{\text{scornerbend}}, A_{\text{scornermin}}) = \mathbf{310 \text{ mm}^2}$$

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

$$b_w = b_{\text{edge}} + (h_{\text{edge}} / \tan(\alpha_{\text{edge}})) / 2 = \mathbf{530 \text{ mm}}$$

Applied shear stress

$$v_{\text{corner}} = V_{\Sigma\text{corner}} / (b_w \times d_{\text{edgetop}}) = \mathbf{0.193 \text{ N/mm}^2}$$

Tension steel ratio

$$\rho_{\text{corner}} = 100 \times A_{\text{sedgetop}} / (b_w \times d_{\text{edgetop}}) = \mathbf{0.282}$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$v_{\text{ccorner}} = \mathbf{0.462 \text{ N/mm}^2}$$

$v_{\text{corner}} \leq v_{\text{ccorner}} + 0.4 \text{ N/mm}^2$ - Therefore minimum links required

Link area to spacing ratio required

$$A_{\text{sv_upon_Svreqcorner}} = 0.4 \text{ N/mm}^2 \times b_w / ((1.0/\gamma_s) \times f_{ys}) = \mathbf{0.488 \text{ mm}}$$

Link area to spacing ratio provided

$$A_{\text{sv_upon_Svprovedge}} = N_{\text{edgelink}} \times \pi \times \phi_{\text{edgelink}}^2 / (4 \times S_{\text{vedge}}) = \mathbf{0.628 \text{ mm}}$$

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

$$\text{Ratio}_{\text{basiccorner}} = \mathbf{7.0}$$

Moment factor

$$M_{\text{factorcorner}} = M_{\Sigma\text{corner}} / (b_{\text{edgetop}} \times d_{\text{edgetop}}^2) = \mathbf{0.501 \text{ N/mm}^2}$$

Steel service stress

$$f_{\text{scorner}} = 2/3 \times f_y \times A_{\text{scornerbend}} / A_{\text{sedgetop}} = \mathbf{162.338 \text{ N/mm}^2}$$

Modification factor

$$MF_{\text{corner}} = \min(2.0, 0.55 + [(477 \text{ N/mm}^2 - f_{\text{scorner}}) / (120 \times (0.9 \text{ N/mm}^2 + M_{\text{factorcorner}}))])$$

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

Modified allowable span to depth ratio

$$\text{Ratio}_{\text{allowcorner}} = \text{Ratio}_{\text{basiccorner}} \times MF_{\text{corner}} = \mathbf{14.000}$$

Actual span to depth ratio

$$\text{Ratio}_{\text{actualcorner}} = l_{\text{corner}} / d_{\text{edgetop}} = \mathbf{6.363}$$

PASS - $\text{Ratio}_{\text{actualcorner}} \leq \text{Ratio}_{\text{allowcorner}}$ - Edge beam span to depth ratio is adequate