

Title: Structural Calculations

**Project: New House at High Street,
 Trelewis**

Client: Northrise

Document No: 001
Project No: 18300
Prepared By: AS
Checked By: MJ

Revision: A
Date: 24/09/2024

DESIGN STATEMENT

The design has been done in accordance with British Standards BS6399, BS5628 (Masonry), BS5268 (Timber), BS 5950 (Steel), & BS8110-1:1997 (Foundations).

Information provided by the client.

The design is for 1No. Steel Beam supporting timber first-floor joists, 1No. Steel Beam supporting balcony/terrace timber floor joists and masonry bearing over the sliding doors, 1No. Steel Channel supporting balcony timber floor joists, 1No. Pad Foundation supporting balcony steel posts, 1No. Steel Beam supporting masonry gable over the sliding doors, 1No. Timber first-floor joists, 1No. Timber roof rafters supporting proposed roof structure over the first-floor bedroom, 1No. Roof support Steel Beam supporting timber ridge beam, 2No. Strip Foundation design analysis, one design to supporting masonry gable wall and another design analysis to support front/rear elevation masonry walls, 1No. Steel Beam supporting garage roof timber joists, 1No. Steel Beam supporting roof steel beam over the garage doors, 1No. Timber flat roof joists supporting garage roof, and 1No. RC Masonry Retaining Wall supporting proposed terrace structure.

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 100kN/m², to be confirmed on site.

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

Refer to our sketches 18300 – SK01, SK02, SK03, SK04, SK05 for references.

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

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.

CDM Comments:

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

 Vale Consultancy CONSULTING CIVIL & STRUCTURAL ENGINEERS 29 Bocam Park Old Field Road, Pencoed CF35 5LJ	Project				Job Ref.	
	New House at High Street, Trelewis				18300	
	Section				Sheet no./rev.	
	Design loadings				03	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	AS	12/02/2024	MJ	12/02/2024	MJ	12/02/2024

New House at High Street, Trelewis

Steel design calculations have been completed in accordance with (BS5950).

Timber design calculations have been completed in accordance with (BS5268-2:2002).

Masonry design calculations have been completed in accordance with (BS5628-1:2005).

Foundations design calculations have been completed in accordance with (BS8110-1:1997).

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.

Design Loadings

1. Roof 35° Pitch (Max)

Dead Loading:		
Tiles	=	0.45 kN/m ²
Insulation, Felt & Battens	=	0.10 kN/m ²
Truss or Purlins/Rafters	=	0.20 kN/m ²
Plasterboard & skim	=	0.15 kN/m ²
		<u>0.90 kN/m² /cos(35) = 1.10kN/m² Total Dead Loading</u>

Imposed Loading:		
Snow	=	<u>0.60 kN/m² Total Live Loading</u>

2. Roof Flat

Dead Loading:		
Waterproof Single Ply Membrane	=	0.20 kN/m ²
Plyboard	=	0.25 kN/m ²
Joists	=	0.15 kN/m ²
Insulation	=	0.05 kN/m ²
Plasterboard & Skim	=	0.15 kN/m ²
		<u>0.80 kN/m² Total Dead Loading</u>

Imposed Loading:		
Snow	=	<u>0.60 kN/m² Total Live Loading</u>

3. Walls

Dead Loading:		
Render	=	0.15 kN/m ²
200mm masonry	=	4.30kN/m ²
Plaster	=	0.15 kN/m ²
		<u>4.60 kN/m² Total Dead Loading</u>

4. Residential Floor

Dead Loading:		
Boarding	=	0.15 kN/m ²
Insulation	=	0.05 kN/m ²
Joists	=	0.15 kN/m ²
Plaster and skim Ceiling	=	0.15 kN/m ²
		<u>0.5 kN/m² Total Dead Loading</u>

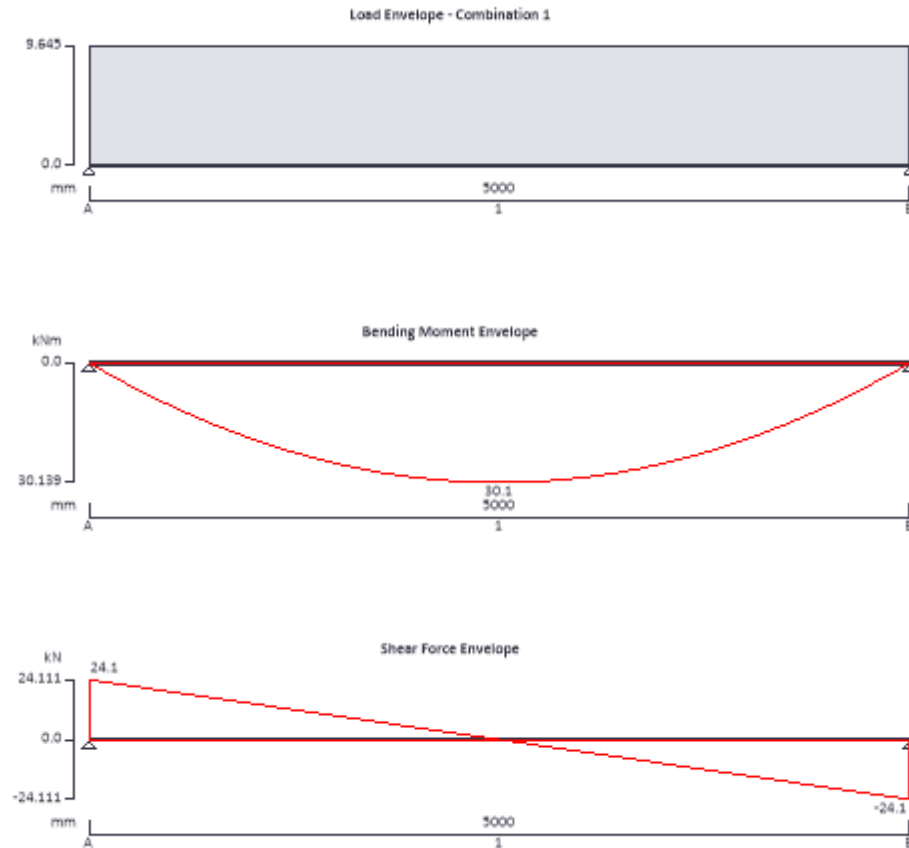
Imposed Loading:		
Residential	=	<u>1.50 kN/m² Total Live Loading</u>

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Steel Beam supporting timber first-floor joists				Start page no./Revision 04	
Calcs by AS	Calcs date 13/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained
Rotationally free

Support B

Vertically restrained
Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$
Floor - Dead full UDL 1.5 kN/m
Floor - Imposed full UDL 4.5 kN/m

Load combinations

Load combination 1

Support A

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

Support B

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

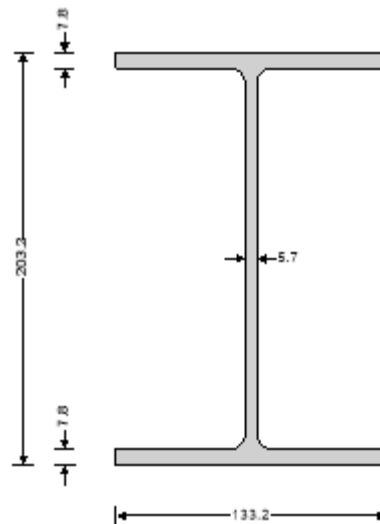
Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Steel Beam supporting timber first-floor joists				Start page no./Revision 05	
Calcs by AS	Calcs date 13/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

Analysis results

Maximum moment	$M_{max} = 30.1$ kNm	$M_{min} = 0$ kNm
Maximum shear	$V_{max} = 24.1$ kN	$V_{min} = -24.1$ kN
Deflection	$\delta_{max} = 10.6$ mm	$\delta_{min} = 0$ mm
Maximum reaction at support A	$R_{A_max} = 24.1$ kN	$R_{A_min} = 24.1$ kN
Unfactored dead load reaction at support A	$R_{A_Dead} = 4.4$ kN	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 11.3$ kN	
Maximum reaction at support B	$R_{B_max} = 24.1$ kN	$R_{B_min} = 24.1$ kN
Unfactored dead load reaction at support B	$R_{B_Dead} = 4.4$ kN	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 11.3$ kN	

Section details

Section type	UB 203x133x25 (BS4-1)
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 7.8$ mm
Design strength	$p_y = 275$ N/mm ²
Modulus of elasticity	$E = 205000$ N/mm ²



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT,A} = 1.00$
	$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$$

Internal compression parts - Table 11

Depth of section	$d = 172.4$ mm	
	$d / t = 30.2 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Steel Beam supporting timber first-floor joists				Start page no./Revision 06	
Calcs by AS	Calcs date 13/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

Outstand flanges - Table 11

Width of section

$$b = B / 2 = 66.6 \text{ mm}$$

$$b / T = 8.5 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 24.1 \text{ kN}$$

$$d / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1158 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 191.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 30.1 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 70.9 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 13.889 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 10.596 \text{ mm}$$

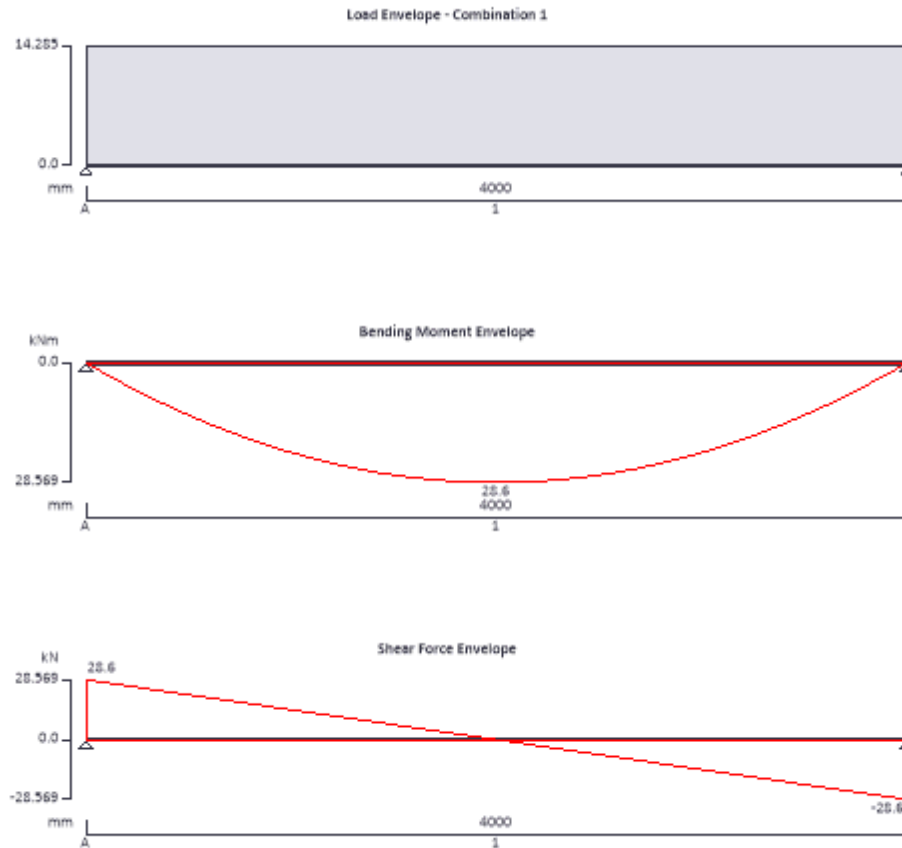
PASS - Maximum deflection does not exceed deflection limit

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Steel Beam supporting masonry bearing over the sliding doors				Start page no./Revision 07	
Calcs by AS	Calcs date 13/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained
Rotationally free

Support B

Vertically restrained
Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$
First-Floor - Dead full UDL 0.5 kN/m
First-Floor - Imposed full UDL 1.5 kN/m
Terrace Floor - Dead full UDL 1.3 kN/m
Terrace Floor - Imposed full UDL 2.4 kN/m
Masonry - Dead full UDL 3.7 kN/m

Load combinations

Load combination 1

Support A

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

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Steel Beam supporting masonry bearing over the sliding doors				Start page no./Revision 08	
Calcs by AS	Calcs date 13/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

Support B

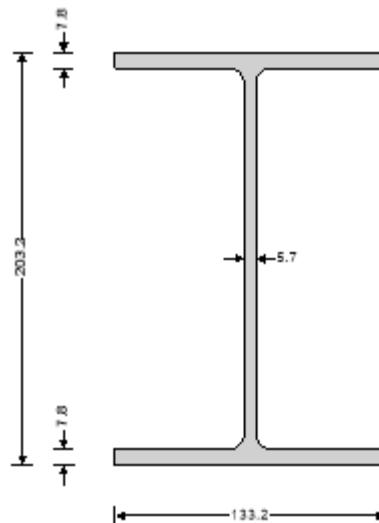
Dead $\times 1.40$
Imposed $\times 1.60$

Analysis results

Maximum moment	$M_{max} = 28.6 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 28.6 \text{ kN}$	$V_{min} = -28.6 \text{ kN}$
Deflection	$\delta_{max} = 6.7 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 28.6 \text{ kN}$	$R_{A_{min}} = 28.6 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{Dead}} = 11.5 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{Imposed}} = 7.8 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 28.6 \text{ kN}$	$R_{B_{min}} = 28.6 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{Dead}} = 11.5 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{Imposed}} = 7.8 \text{ kN}$	

Section details

Section type	UB 203x133x25 (BS4-1)
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 7.8 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT.A} = 1.00$
	$K_{LT.B} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$$

Internal compression parts - Table 11

Depth of section	$d = 172.4 \text{ mm}$	
	$d / t = 30.2 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

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Calcs for Steel Beam supporting masonry bearing over the sliding doors				Start page no./Revision 09	
Calcs by AS	Calcs date 13/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

Outstand flanges - Table 11

Width of section

$$b = B / 2 = 66.6 \text{ mm}$$

$$b / T = 8.5 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 28.6 \text{ kN}$$

$$d / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1158 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 191.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 28.6 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 70.9 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 11.111 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 6.702 \text{ mm}$$

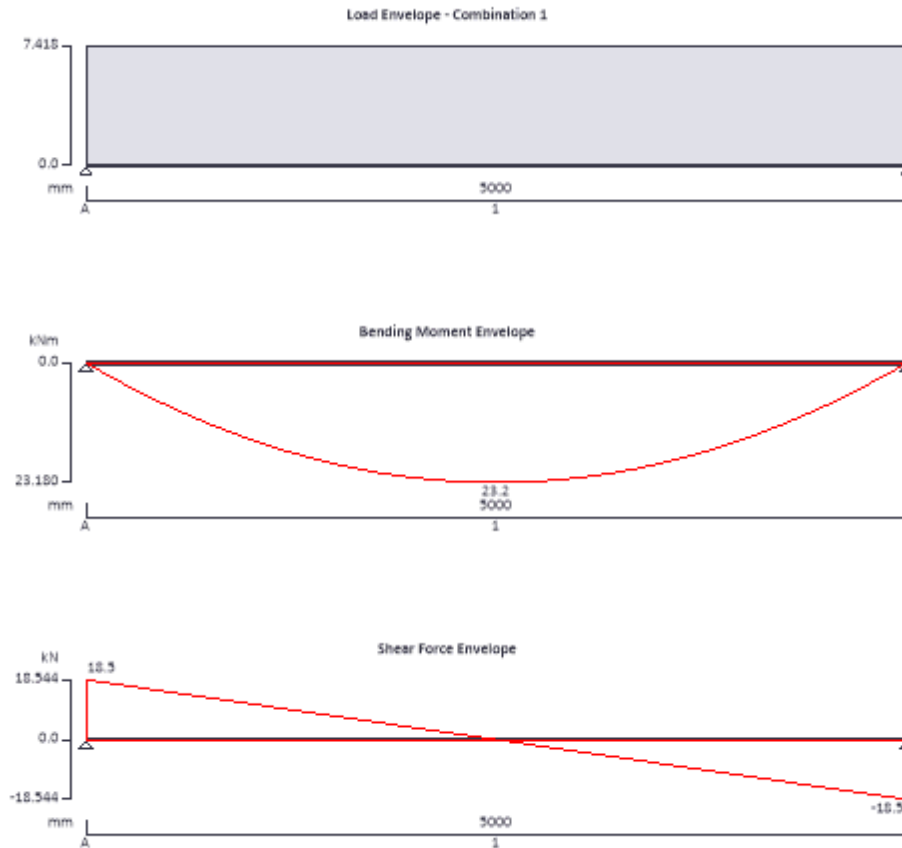
PASS - Maximum deflection does not exceed deflection limit

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Steel Channel supporting terrace/balcony floor				Start page no./Revision 10	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

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

Applied loading

Beam loads	Dead self weight of beam $\times 1$ Terrace Floor - Dead full UDL 1.3 kN/m Terrace Floor - Imposed full UDL 2.4 kN/m Balustrade - Dead full UDL 1 kN/m
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Load combinations

Load combination 1	Support A	Dead $\times 1.40$
		Imposed $\times 1.60$
	Support B	Dead $\times 1.40$
		Imposed $\times 1.60$

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Steel Channel supporting terrace/balcony floor				Start page no./Revision 11	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

Analysis results

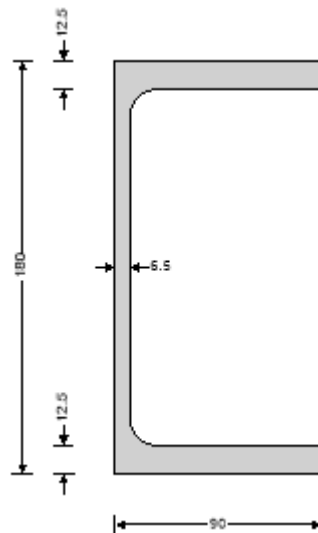
Maximum moment	$M_{max} = 23.2 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 18.5 \text{ kN}$	$V_{min} = -18.5 \text{ kN}$
Deflection	$\delta_{max} = 10.8 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 18.5 \text{ kN}$	$R_{A_min} = 18.5 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 6.4 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 6 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 18.5 \text{ kN}$	$R_{B_min} = 18.5 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 6.4 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 6 \text{ kN}$	

Section details

Section type	UKPFC 180x90x26 (Tata Steel Advance)
Steel grade	S275

From table 9: Design strength p_y

Thickness of element	$\max(T, t) = 12.5 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT,A} = 1.00$
	$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{275 \text{ N/mm}^2 / p_y} = 1.00$$

Internal compression parts - Table 11

Depth of section	$d = 131 \text{ mm}$	
	$d / t = 20.2 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Steel Channel supporting terrace/balcony floor				Start page no./Revision 12	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

Outstand flanges - Table 11

Width of section

$$b = B = 90 \text{ mm}$$

$$b / T = 7.2 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 18.5 \text{ kN}$$

$$d / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1170 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 193.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 23.2 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 63.8 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 13.889 \text{ mm}$$

Maximum deflection span 1

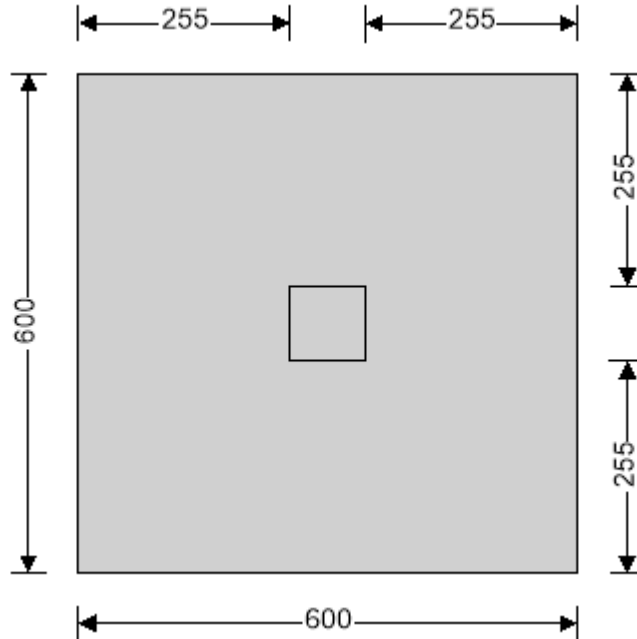
$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 10.824 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Pad Foundation supporting terrace steel posts				Start page no./Revision 13	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

PAD FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07



Pad footing details

Length of pad footing	$L = 600 \text{ mm}$
Width of pad footing	$B = 600 \text{ mm}$
Area of pad footing	$A = L \times B = 0.360 \text{ m}^2$
Depth of pad footing	$h = 300 \text{ mm}$
Depth of soil over pad footing	$h_{\text{soil}} = 500 \text{ mm}$
Density of concrete	$\rho_{\text{conc}} = 23.6 \text{ kN/m}^3$

Column details

Column base length	$l_A = 90 \text{ mm}$
Column base width	$b_A = 90 \text{ mm}$
Column eccentricity in x	$e_{Px_A} = 0 \text{ mm}$
Column eccentricity in y	$e_{Py_A} = 0 \text{ mm}$

Soil details

Density of soil	$\rho_{\text{soil}} = 18.0 \text{ kN/m}^3$
Design shear strength	$\phi' = 25.0 \text{ deg}$
Design base friction	$\delta = 19.3 \text{ deg}$
Allowable bearing pressure	$P_{\text{bearing}} = 100 \text{ kN/m}^2$

Axial loading on column

Dead axial load on column	$P_{GA} = 12.8 \text{ kN}$
Imposed axial load on column	$P_{QA} = 12.0 \text{ kN}$
Wind axial load on column	$P_{WA} = 0.0 \text{ kN}$
Total axial load on column	$P_A = 24.8 \text{ kN}$

Foundation loads

Dead surcharge load	$F_{G_{\text{sur}}} = 0.000 \text{ kN/m}^2$
Imposed surcharge load	$F_{Q_{\text{sur}}} = 0.000 \text{ kN/m}^2$

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Pad Foundation supporting terrace steel posts				Start page no./Revision 14	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

Pad footing self weight

$$F_{swt} = h \times \rho_{conc} = \mathbf{7.080 \text{ kN/m}^2}$$

Soil self weight

$$F_{soil} = h_{soil} \times \rho_{soil} = \mathbf{9.000 \text{ kN/m}^2}$$

Total foundation load

$$F = A \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = \mathbf{5.8 \text{ kN}}$$

Calculate pad base reaction

Total base reaction

$$T = F + P_A = \mathbf{30.6 \text{ kN}}$$

Eccentricity of base reaction in x

$$e_{Tx} = (P_A \times e_{Px} + M_{xA} + H_{xA} \times h) / T = \mathbf{0 \text{ mm}}$$

Eccentricity of base reaction in y

$$e_{Ty} = (P_A \times e_{Py} + M_{yA} + H_{yA} \times h) / T = \mathbf{0 \text{ mm}}$$

Check pad base reaction eccentricity

$$\text{abs}(e_{Tx}) / L + \text{abs}(e_{Ty}) / B = \mathbf{0.000}$$

Base reaction acts within middle third of base

Calculate pad base pressures

$$q_1 = T / A - 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = \mathbf{84.969 \text{ kN/m}^2}$$

$$q_2 = T / A - 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = \mathbf{84.969 \text{ kN/m}^2}$$

$$q_3 = T / A + 6 \times T \times e_{Tx} / (L \times A) - 6 \times T \times e_{Ty} / (B \times A) = \mathbf{84.969 \text{ kN/m}^2}$$

$$q_4 = T / A + 6 \times T \times e_{Tx} / (L \times A) + 6 \times T \times e_{Ty} / (B \times A) = \mathbf{84.969 \text{ kN/m}^2}$$

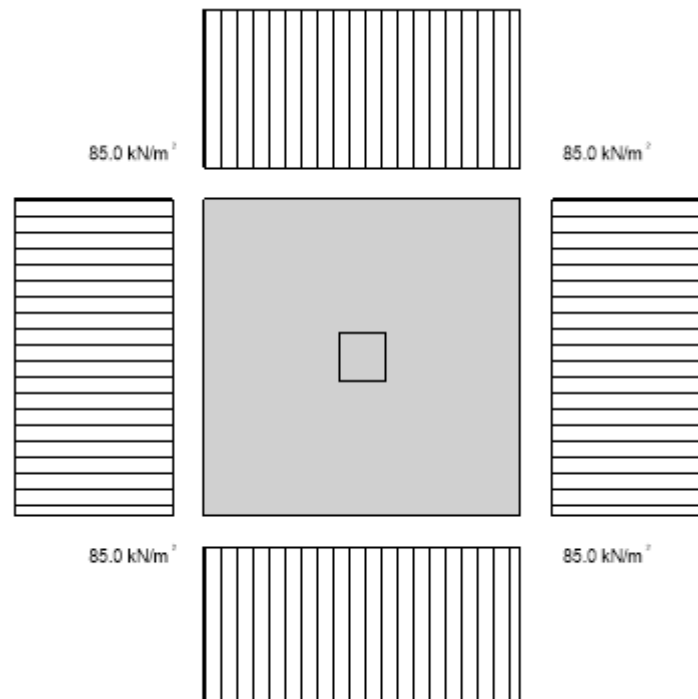
Minimum base pressure

$$q_{min} = \min(q_1, q_2, q_3, q_4) = \mathbf{84.969 \text{ kN/m}^2}$$

Maximum base pressure

$$q_{max} = \max(q_1, q_2, q_3, q_4) = \mathbf{84.969 \text{ kN/m}^2}$$

PASS - Maximum base pressure is less than allowable bearing pressure



Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Timber first-floor joists				Start page no./Revision 15	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

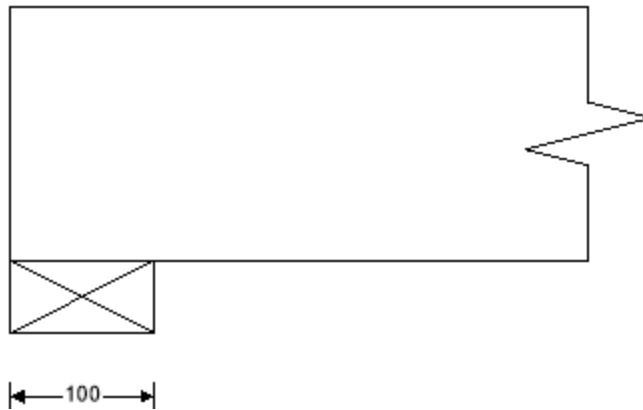
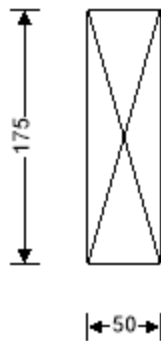
Joist details

Joist breadth	b = 50 mm
Joist depth	h = 175 mm
Joist spacing	s = 400 mm
Timber strength class	C24
Service class of timber	1



Span details

Number of spans	N_{span} = 1
Length of bearing	L_b = 100 mm
Effective length of span	L_{s1} = 3900 mm



Section properties

Second moment of area	$I = b \times h^3 / 12 = 22330729 \text{ mm}^4$
Section modulus	$Z = b \times h^2 / 6 = 255208 \text{ mm}^3$

Loading details

Joist self weight	$F_{swt} = b \times h \times \rho_{char} \times g_{acc} = 0.03 \text{ kN/m}$
Dead load	$F_{d_udl} = 0.50 \text{ kN/m}^2$
Imposed UDL(Long term)	$F_{i_udl} = 1.50 \text{ kN/m}^2$
Imposed point load (Medium term)	$F_{i_pt} = 1.40 \text{ kN}$

Modification factors

Service class for bending parallel to grain	K_{2m} = 1.00
Service class for compression	K_{2c} = 1.00
Service class for shear parallel to grain	K_{2s} = 1.00
Service class for modulus of elasticity	K_{2e} = 1.00

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Section depth factor $K_7 = 1.06$

Load sharing factor $K_8 = 1.10$

Consider long term loads

Load duration factor $K_3 = 1.00$

Maximum bending moment $M = 1.578$ kNm

Maximum shear force $V = 1.619$ kN

Maximum support reaction $R = 1.619$ kN

Maximum deflection $\delta = 10.688$ mm

Check bending stress

Bending stress $\sigma_m = 7.500$ N/mm²

Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 8.754$ N/mm²

Applied bending stress $\sigma_{m_max} = M / Z = 6.184$ N/mm²

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress $\tau = 0.710$ N/mm²

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.781$ N/mm²

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h) = 0.277$ N/mm²

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) $\sigma_{cp1} = 2.400$ N/mm²

Permissible bearing stress $\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 2.640$ N/mm²

Applied bearing stress $\sigma_{c_max} = R / (b \times L_b) = 0.324$ N/mm²

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 11.700$ mm

Bending deflection (based on E_{mean}) $\delta_{bending} = 10.367$ mm

Shear deflection $\delta_{shear} = 0.321$ mm

Total deflection $\delta = \delta_{bending} + \delta_{shear} = 10.688$ mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor $K_3 = 1.25$

Maximum bending moment $M = 1.802$ kNm

Maximum shear force $V = 1.849$ kN

Maximum support reaction $R = 1.849$ kN

Maximum deflection $\delta = 10.413$ mm

Check bending stress

Bending stress $\sigma_m = 7.500$ N/mm²

Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 10.942$ N/mm²

Applied bending stress $\sigma_{m_max} = M / Z = 7.062$ N/mm²

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress $\tau = 0.710$ N/mm²

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.976$ N/mm²

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h) = 0.317$ N/mm²

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PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)

$$\sigma_{cp1} = \mathbf{2.400 \text{ N/mm}^2}$$

Permissible bearing stress

$$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = \mathbf{3.300 \text{ N/mm}^2}$$

Applied bearing stress

$$\sigma_{c_max} = R / (b \times L_b) = \mathbf{0.370 \text{ N/mm}^2}$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = \mathbf{11.700 \text{ mm}}$$

Bending deflection (based on E_{mean})

$$\delta_{bending} = \mathbf{10.047 \text{ mm}}$$

Shear deflection

$$\delta_{shear} = \mathbf{0.366 \text{ mm}}$$

Total deflection

$$\delta = \delta_{bending} + \delta_{shear} = \mathbf{10.413 \text{ mm}}$$

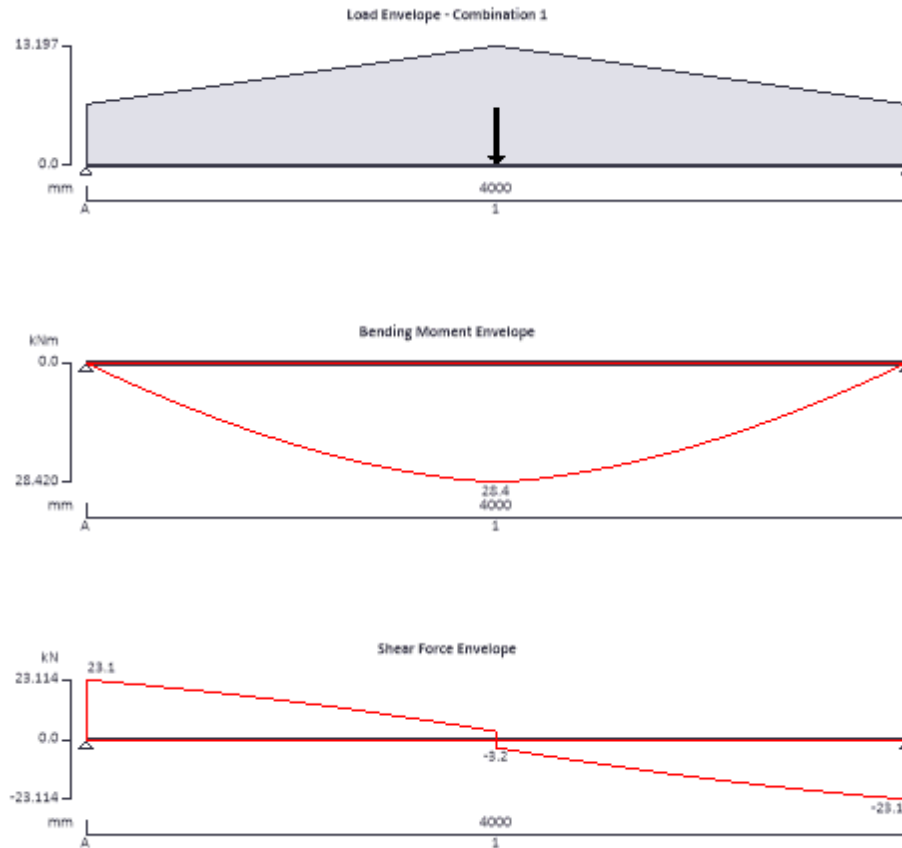
PASS - Actual deflection within permissible limits

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STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained
Rotationally free

Support B

Vertically restrained
Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Masonry Gable - Dead partial VDL 4.6 kN/m at 0 mm to 9.2 kN/m at 2000 mm

Masonry Gable - Dead partial VDL 9.2 kN/m at 2000 mm to 4.6 kN/m at 4000 mm

Timber Ridge - Dead point load 2.8 kN at 2000 mm

Timber Ridge - Imposed point load 1.5 kN at 2000 mm

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

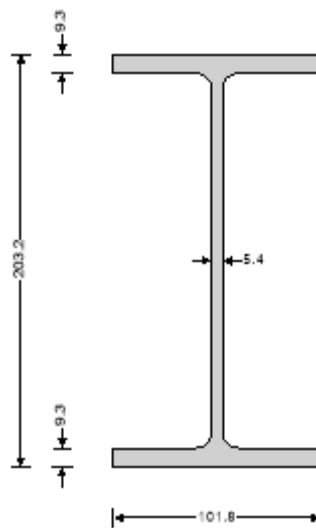
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Analysis results

Maximum moment	$M_{max} = 28.4 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 23.1 \text{ kN}$	$V_{min} = -23.1 \text{ kN}$
Deflection	$\delta_{max} = 7.3 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 23.1 \text{ kN}$	$R_{A_min} = 23.1 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 15.7 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 0.8 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 23.1 \text{ kN}$	$R_{B_min} = 23.1 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 15.7 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 0.8 \text{ kN}$	

Section details

Section type	UB 203x102x23 (BS4-1)
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 9.3 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT,A} = 1.00$
	$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{275 \text{ N/mm}^2 / p_y} = 1.00$$

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Internal compression parts - Table 11

Depth of section $d = 169.4$ mm
 $d / t = 31.4 \times \epsilon \leq 80 \times \epsilon$ Class 1 plastic

Outstand flanges - Table 11

Width of section $b = B / 2 = 50.9$ mm
 $b / T = 5.5 \times \epsilon \leq 9 \times \epsilon$ Class 1 plastic
Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 23.1$ kN
 $d / t < 70 \times \epsilon$
Web does not need to be checked for shear buckling
 Shear area $A_v = t \times D = 1097$ mm²
 Design shear resistance $P_v = 0.6 \times p_y \times A_v = 181.1$ kN
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 28.4$ kNm
 Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 64.4$ kNm
PASS - Moment capacity exceeds design bending moment

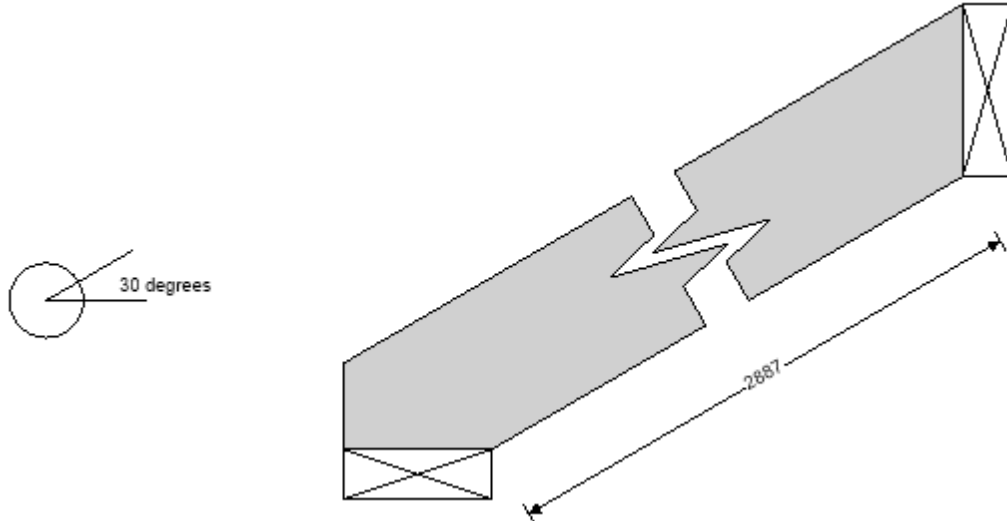
Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads
 Limiting deflection $\delta_{lim} = L_{s1} / 360 = 11.111$ mm
 Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 7.331$ mm
PASS - Maximum deflection does not exceed deflection limit

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TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections
Depth of timber sections
Rafter spacing
Rafter slope
Clear span of rafter on horizontal
Clear span of rafter on slope
Rafter span
Timber strength class

$b = 50 \text{ mm}$
 $h = 150 \text{ mm}$
 $s = 400 \text{ mm}$
 $\alpha = 30.0 \text{ deg}$
 $L_{clh} = 2500 \text{ mm}$
 $L_{cl} = L_{clh} / \cos(\alpha) = 2887 \text{ mm}$
Single span
C24

Section properties

Cross sectional area of rafter
Section modulus
Second moment of area
Radius of gyration

$A = b \times h = 7500 \text{ mm}^2$
 $Z = b \times h^2 / 6 = 187500 \text{ mm}^3$
 $I = b \times h^3 / 12 = 14062500 \text{ mm}^4$
 $r = \sqrt{I / A} = 43.3 \text{ mm}$

Loading details

Rafter self weight
Dead load on slope
Imposed load on plan
Imposed point load

$F_j = b \times h \times \rho_{char} \times g_{acc} = 0.03 \text{ kN/m}$
 $F_d = 0.90 \text{ kN/m}^2$
 $F_u = 0.60 \text{ kN/m}^2$
 $F_p = 0.90 \text{ kN}$

Modification factors

Section depth factor
Load sharing factor

$K_7 = (300 \text{ mm} / h)^{0.11} = 1.08$
 $K_8 = 1.10$

Consider long term load condition

Load duration factor
Total UDL perpendicular to rafter
Notional bearing length
Effective span

$K_3 = 1.00$
 $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = 0.334 \text{ kN/m}$
 $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 4 \text{ mm}$
 $L_{eff} = L_{cl} + L_b = 2890 \text{ mm}$

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Check bending stress

Bending stress parallel to grain

$$\sigma_m = 7.500 \text{ N/mm}^2$$

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 8.904 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 1.861 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain

$$\sigma_c = 7.900 \text{ N/mm}^2$$

Minimum modulus of elasticity

$$E_{min} = 7200 \text{ N/mm}^2$$

Compression member factor

$$K_{12} = 0.62$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 5.373 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.223 \text{ N/mm}^2$$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 15.948 \text{ N/mm}^2$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.987$$

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.253 < 1$$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain

$$\tau = 0.710 \text{ N/mm}^2$$

Permissible shear stress

$$\tau_{adm} = \tau \times K_3 \times K_8 = 0.781 \text{ N/mm}^2$$

Applied shear stress

$$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = 0.097 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = 8.671 \text{ mm}$$

Bending deflection

$$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = 1.999 \text{ mm}$$

Shear deflection

$$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = 0.083 \text{ mm}$$

Total deflection

$$\delta_{max} = \delta_b + \delta_s = 2.082 \text{ mm}$$

PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor

$$K_3 = 1.25$$

Total UDL perpendicular to rafter

$$F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) = 0.514 \text{ kN/m}$$

Notional bearing length

$$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 6 \text{ mm}$$

Effective span

$$L_{eff} = L_{cl} + L_b = 2892 \text{ mm}$$

Check bending stress

Bending stress parallel to grain

$$\sigma_m = 7.500 \text{ N/mm}^2$$

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 11.130 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 2.867 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain

$$\sigma_c = 7.900 \text{ N/mm}^2$$

Minimum modulus of elasticity

$$E_{min} = 7200 \text{ N/mm}^2$$

Compression member factor

$$K_{12} = 0.58$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 6.298 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.343 \text{ N/mm}^2$$

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PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{15.926 \text{ N/mm}^2}$
Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = \mathbf{0.981}$
Combined axial compression and bending check $\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = \mathbf{0.317 < 1}$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain $\tau = \mathbf{0.710 \text{ N/mm}^2}$
Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{0.976 \text{ N/mm}^2}$
Applied shear stress $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = \mathbf{0.149 \text{ N/mm}^2}$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{eff} = \mathbf{8.677 \text{ mm}}$
Bending deflection $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = \mathbf{3.085 \text{ mm}}$
Shear deflection $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = \mathbf{0.127 \text{ mm}}$
Total deflection $\delta_{max} = \delta_b + \delta_s = \mathbf{3.212 \text{ mm}}$

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor $K_3 = \mathbf{1.50}$
Total UDL perpendicular to rafter $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = \mathbf{0.334 \text{ kN/m}}$
Notional bearing length $L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{7 \text{ mm}}$
Effective span $L_{eff} = L_{cl} + L_b = \mathbf{2893 \text{ mm}}$

Check bending stress

Bending stress parallel to grain $\sigma_m = \mathbf{7.500 \text{ N/mm}^2}$
Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{13.355 \text{ N/mm}^2}$
Applied bending stress $\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) + F_p \times \cos(\alpha) \times L_{eff} / (4 \times Z) = \mathbf{4.871 \text{ N/mm}^2}$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain $\sigma_c = \mathbf{7.900 \text{ N/mm}^2}$
Minimum modulus of elasticity $E_{min} = \mathbf{7200 \text{ N/mm}^2}$
Compression member factor $K_{12} = \mathbf{0.54}$
Permissible compressive stress $\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{7.062 \text{ N/mm}^2}$
Applied compressive stress $\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) + F_p \times \sin(\alpha) / A = \mathbf{0.283 \text{ N/mm}^2}$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{15.916 \text{ N/mm}^2}$
Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = \mathbf{0.986}$
Combined axial compression and bending check $\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = \mathbf{0.410 < 1}$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain $\tau = \mathbf{0.710 \text{ N/mm}^2}$
Permissible shear stress $\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{1.172 \text{ N/mm}^2}$
Applied shear stress $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A) = \mathbf{0.253 \text{ N/mm}^2}$

PASS - Applied shear stress within permissible limits

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Check deflection

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = \mathbf{8.680 \text{ mm}}$$

Bending deflection

$$\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha) / 48) / (E_{mean} \times I) = \mathbf{4.597 \text{ mm}}$$

Shear deflection

$$\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{mean} \times A) = \mathbf{0.217 \text{ mm}}$$

Total deflection

$$\delta_{max} = \delta_b + \delta_s = \mathbf{4.813 \text{ mm}}$$

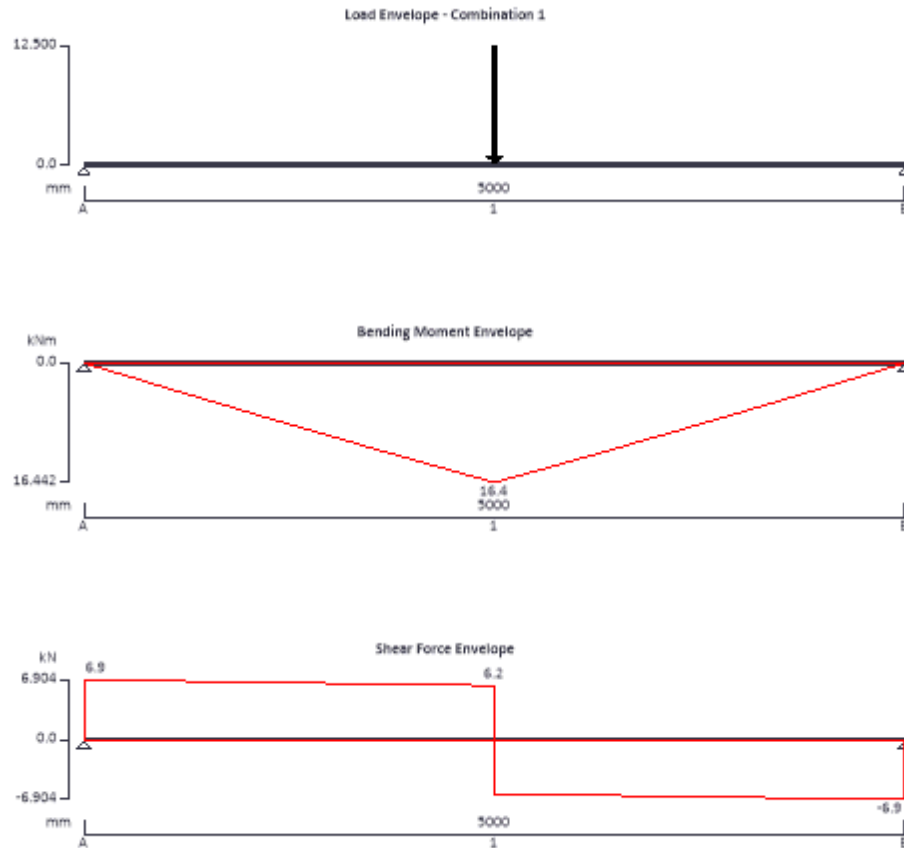
PASS - Total deflection within permissible limits

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STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained
Rotationally free

Support B

Vertically restrained
Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$
Timber Ridge - Dead point load 5.5 kN at 2500 mm
Timber Ridge - Imposed point load 3 kN at 2500 mm

Load combinations

Load combination 1

Support A

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

Support B

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

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Analysis results

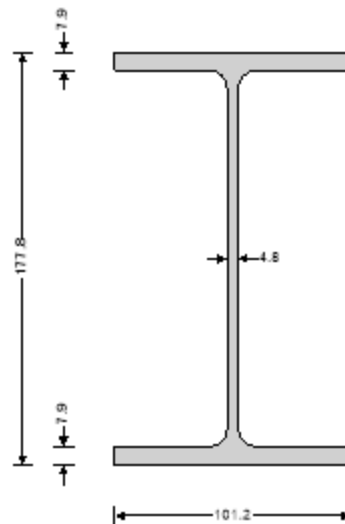
Maximum moment	$M_{max} = 16.4 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 6.9 \text{ kN}$	$V_{min} = -6.9 \text{ kN}$
Deflection	$\delta_{max} = 8.5 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 6.9 \text{ kN}$	$R_{A_min} = 6.9 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 3.2 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 1.5 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 6.9 \text{ kN}$	$R_{B_min} = 6.9 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 3.2 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 1.5 \text{ kN}$	

Section details

Section type	UB 178x102x19 (BS4-1)
Steel grade	S275

From table 9: Design strength p_y

Thickness of element	$\max(T, t) = 7.9 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT,A} = 1.00$
	$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{275 \text{ N/mm}^2 / p_y} = 1.00$$

Internal compression parts - Table 11

Depth of section	$d = 146.8 \text{ mm}$	
	$d / t = 30.6 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Roof support Steel Beam				Start page no./Revision 27	
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Outstand flanges - Table 11

Width of section

$$b = B / 2 = \mathbf{50.6 \text{ mm}}$$

$$b / T = 6.4 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{6.9 \text{ kN}}$$

$$d / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = \mathbf{853 \text{ mm}^2}$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = \mathbf{140.8 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{16.4 \text{ kNm}}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{47.1 \text{ kNm}}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{13.889 \text{ mm}}$$

Maximum deflection span 1

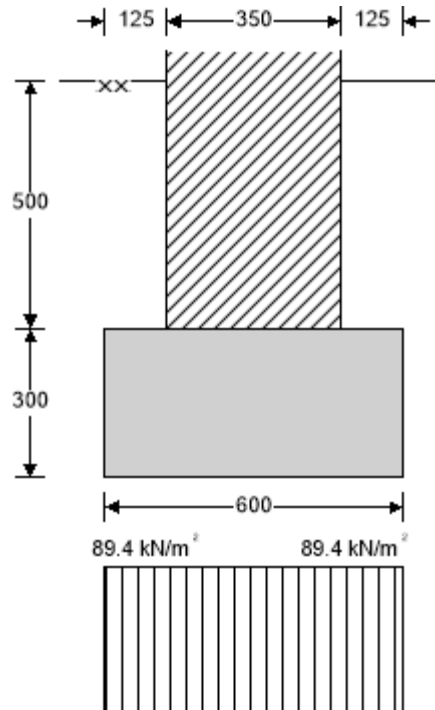
$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{8.51 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Strip Foundation supporting masonry gable wall				Start page no./Revision 28	
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STRIP FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07



Strip footing details

Width of strip footing
Depth of strip footing
Depth of soil over strip footing
Density of concrete

$B = 600$ mm
 $h = 300$ mm
 $h_{\text{soil}} = 500$ mm
 $\rho_{\text{conc}} = 23.6$ kN/m³

Load details

Load width
Load eccentricity

$b = 350$ mm
 $e_P = 0$ mm

Soil details

Density of soil
Design shear strength
Design base friction
Allowable bearing pressure

$\rho_{\text{soil}} = 18.0$ kN/m³
 $\phi' = 25.0$ deg
 $\delta = 19.3$ deg
 $P_{\text{bearing}} = 100$ kN/m²

Axial loading on strip footing

Dead axial load
Imposed axial load
Wind axial load
Total axial load

$P_G = 38.0$ kN/m
 $P_Q = 6.0$ kN/m
 $P_W = 0.0$ kN/m
 $P = 44.0$ kN/m

Foundation loads

Dead surcharge load
Imposed surcharge load
Strip footing self weight
Soil self weight

$F_{G_{\text{sur}}} = 0.000$ kN/m²
 $F_{Q_{\text{sur}}} = 0.000$ kN/m²
 $F_{\text{swt}} = h \times \rho_{\text{conc}} = 7.080$ kN/m²
 $F_{\text{soil}} = h_{\text{soil}} \times \rho_{\text{soil}} = 9.000$ kN/m²

Project New House at High Street, Trelewis				Job no. 18300	
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Total foundation load

$$F = B \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = \mathbf{9.6 \text{ kN/m}}$$

Calculate base reaction

Total base reaction

$$T = F + P = \mathbf{53.6 \text{ kN/m}}$$

Eccentricity of base reaction in x

$$e_T = (P \times e_P + M + H \times h) / T = \mathbf{0 \text{ mm}}$$

Base reaction acts within middle third of base

Calculate base pressures

$$q_1 = (T / B) \times (1 - 6 \times e_T / B) = \mathbf{89.413 \text{ kN/m}^2}$$

$$q_2 = (T / B) \times (1 + 6 \times e_T / B) = \mathbf{89.413 \text{ kN/m}^2}$$

Minimum base pressure

$$q_{min} = \min(q_1, q_2) = \mathbf{89.413 \text{ kN/m}^2}$$

Maximum base pressure

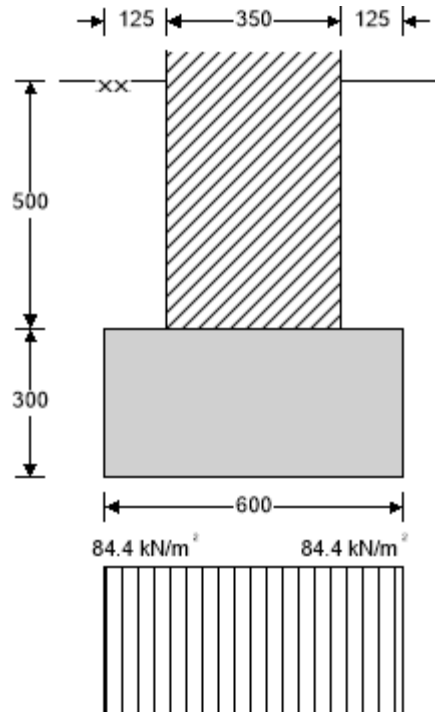
$$q_{max} = \max(q_1, q_2) = \mathbf{89.413 \text{ kN/m}^2}$$

PASS - Maximum base pressure is less than allowable bearing pressure

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Strip Foundation supporting front/rear masonry wall				Start page no./Revision 30	
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STRIP FOOTING ANALYSIS AND DESIGN (BS8110-1:1997)

Tedds calculation version 2.0.07



Strip footing details

Width of strip footing
Depth of strip footing
Depth of soil over strip footing
Density of concrete

$B = 600$ mm
 $h = 300$ mm
 $h_{\text{soil}} = 500$ mm
 $\rho_{\text{conc}} = 23.6$ kN/m³

Load details

Load width
Load eccentricity

$b = 350$ mm
 $e_P = 0$ mm

Soil details

Density of soil
Design shear strength
Design base friction
Allowable bearing pressure

$\rho_{\text{soil}} = 18.0$ kN/m³
 $\phi' = 25.0$ deg
 $\delta = 19.3$ deg
 $P_{\text{bearing}} = 100$ kN/m²

Axial loading on strip footing

Dead axial load
Imposed axial load
Wind axial load
Total axial load

$P_G = 32.0$ kN/m
 $P_Q = 9.0$ kN/m
 $P_W = 0.0$ kN/m
 $P = 41.0$ kN/m

Foundation loads

Dead surcharge load
Imposed surcharge load
Strip footing self weight
Soil self weight

$F_{G\text{sur}} = 0.000$ kN/m²
 $F_{Q\text{sur}} = 0.000$ kN/m²
 $F_{\text{swt}} = h \times \rho_{\text{conc}} = 7.080$ kN/m²
 $F_{\text{soil}} = h_{\text{soil}} \times \rho_{\text{soil}} = 9.000$ kN/m²

Project New House at High Street, Trelewis				Job no. 18300	
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Total foundation load

$$F = B \times (F_{Gsur} + F_{Qsur} + F_{swt} + F_{soil}) = \mathbf{9.6 \text{ kN/m}}$$

Calculate base reaction

Total base reaction

$$T = F + P = \mathbf{50.6 \text{ kN/m}}$$

Eccentricity of base reaction in x

$$e_T = (P \times e_P + M + H \times h) / T = \mathbf{0 \text{ mm}}$$

Base reaction acts within middle third of base

Calculate base pressures

$$q_1 = (T / B) \times (1 - 6 \times e_T / B) = \mathbf{84.413 \text{ kN/m}^2}$$

$$q_2 = (T / B) \times (1 + 6 \times e_T / B) = \mathbf{84.413 \text{ kN/m}^2}$$

Minimum base pressure

$$q_{min} = \min(q_1, q_2) = \mathbf{84.413 \text{ kN/m}^2}$$

Maximum base pressure

$$q_{max} = \max(q_1, q_2) = \mathbf{84.413 \text{ kN/m}^2}$$

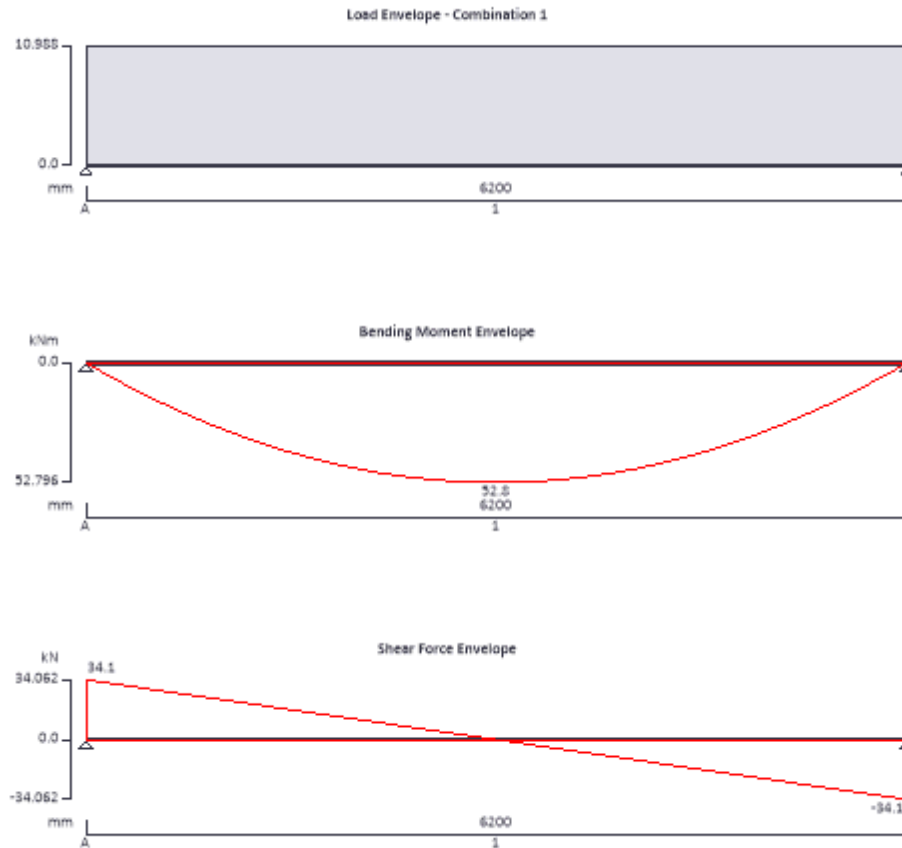
PASS - Maximum base pressure is less than allowable bearing pressure

Project New House at High Street, Trelewis				Job no. 18300	
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Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

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

Applied loading

Beam loads	Dead self weight of beam $\times 1$ Roof - Dead full UDL 2.4 kN/m Roof - Imposed full UDL 4.5 kN/m
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Load combinations

Load combination 1	Support A	Dead $\times 1.40$ Imposed $\times 1.60$
	Support B	Dead $\times 1.40$ Imposed $\times 1.60$

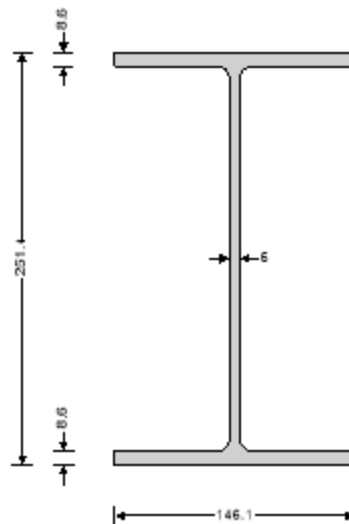
Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Steel Beam supporting garage roof joists				Start page no./Revision 33	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

Analysis results

Maximum moment	$M_{max} = 52.8 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 34.1 \text{ kN}$	$V_{min} = -34.1 \text{ kN}$
Deflection	$\delta_{max} = 15.3 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 34.1 \text{ kN}$	$R_{A_min} = 34.1 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 8.4 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 14 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 34.1 \text{ kN}$	$R_{B_min} = 34.1 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 8.4 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 13.9 \text{ kN}$	

Section details

Section type	UB 254x146x31 (BS4-1)
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 8.6 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT,A} = 1.00$
	$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$$

Internal compression parts - Table 11

Depth of section	$d = 219 \text{ mm}$	
	$d / t = 36.5 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

Project New House at High Street, Trelewis				Job no. 18300	
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Outstand flanges - Table 11

Width of section

$$b = B / 2 = 73.1 \text{ mm}$$

$$b / T = 8.5 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 34.1 \text{ kN}$$

$$d / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1508 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 248.9 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 52.8 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 108.1 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 17.222 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 15.323 \text{ mm}$$

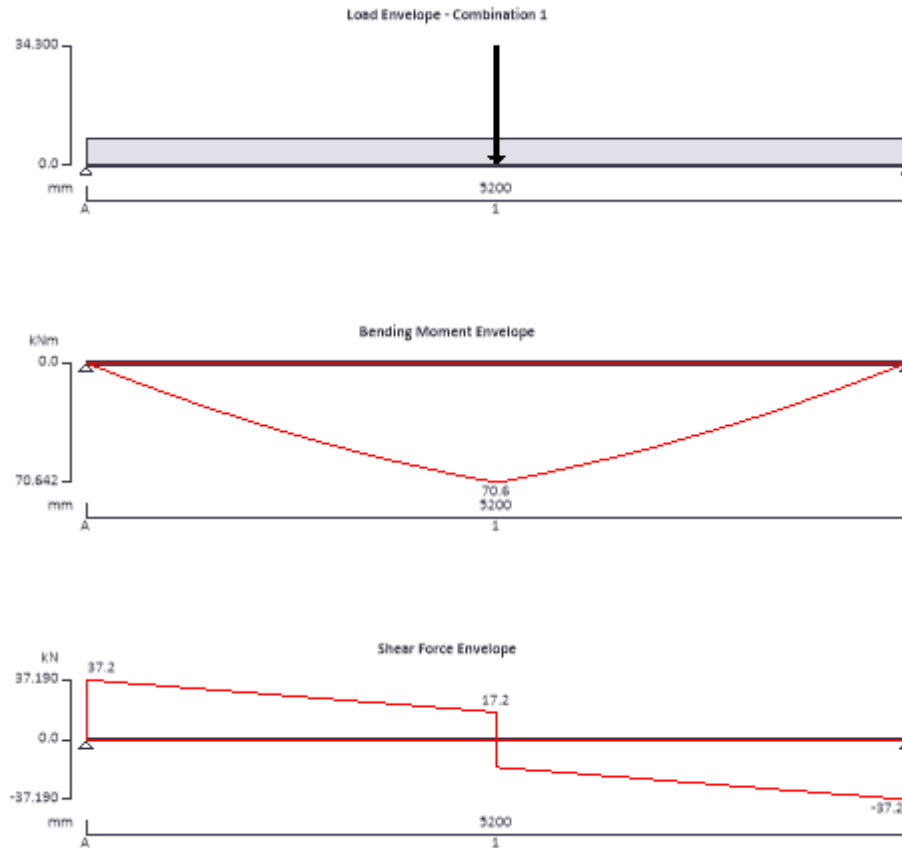
PASS - Maximum deflection does not exceed deflection limit

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Steel Beam supporting garage door				Start page no./Revision 35	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained
Rotationally free

Support B

Vertically restrained
Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$
Steel Beam - Dead point load 8.5 kN at 2600 mm
Steel Beam - Imposed point load 14 kN at 2600 mm
Masonry Bearing - Dead full UDL 5 kN/m
Garage Door - Dead full UDL 0.2 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 $\times 1.40$

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Steel Beam supporting garage door				Start page no./Revision 36	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

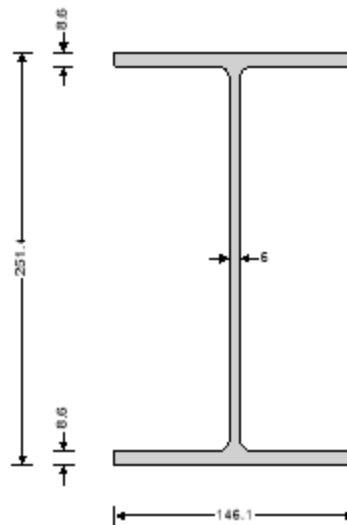
Imposed $\times 1.60$

Analysis results

Maximum moment	$M_{\max} = 70.6 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 37.2 \text{ kN}$	$V_{\min} = -37.2 \text{ kN}$
Deflection	$\delta_{\max} = 13.1 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 37.2 \text{ kN}$	$R_{A_{\min}} = 37.2 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{\text{Dead}}} = 18.6 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{\text{Imposed}}} = 7 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 37.2 \text{ kN}$	$R_{B_{\min}} = 37.2 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{\text{Dead}}} = 18.6 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{\text{Imposed}}} = 7 \text{ kN}$	

Section details

Section type	UB 254x146x31 (BS4-1)
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 8.6 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT,A} = 1.00$
	$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

$$\varepsilon = \sqrt{275 \text{ N/mm}^2 / p_y} = 1.00$$

Internal compression parts - Table 11

Depth of section	$d = 219 \text{ mm}$	
	$d / t = 36.5 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

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Outstand flanges - Table 11

Width of section

$$b = B / 2 = 73.1 \text{ mm}$$

$$b / T = 8.5 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 37.2 \text{ kN}$$

$$d / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1508 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 248.9 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 70.6 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 108.1 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 14.444 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 13.078 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Timber flat roof joists over garage				Start page no./Revision 38	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

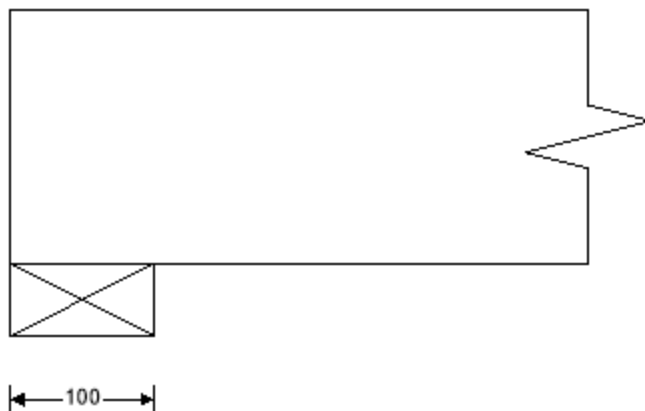
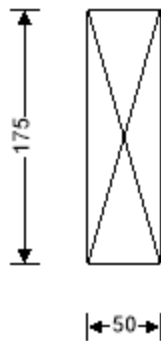
Joist details

Joist breadth	b = 50 mm
Joist depth	h = 175 mm
Joist spacing	s = 400 mm
Timber strength class	C24
Service class of timber	1



Span details

Number of spans	N_{span} = 1
Length of bearing	L_b = 100 mm
Effective length of span	L_{s1} = 3000 mm



Section properties

Second moment of area	$I = b \times h^3 / 12 = 22330729 \text{ mm}^4$
Section modulus	$Z = b \times h^2 / 6 = 255208 \text{ mm}^3$

Loading details

Joist self weight	$F_{swt} = b \times h \times \rho_{char} \times g_{acc} = 0.03 \text{ kN/m}$
Dead load	$F_{d_udl} = 0.80 \text{ kN/m}^2$
Imposed UDL(Long term)	$F_{i_udl} = 1.50 \text{ kN/m}^2$
Imposed point load (Medium term)	$F_{i_pt} = 1.40 \text{ kN}$

Modification factors

Service class for bending parallel to grain	K_{2m} = 1.00
Service class for compression	K_{2c} = 1.00
Service class for shear parallel to grain	K_{2s} = 1.00
Service class for modulus of elasticity	K_{2e} = 1.00

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Timber flat roof joists over garage				Start page no./Revision 39	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

Section depth factor $K_7 = 1.06$

Load sharing factor $K_8 = 1.10$

Consider long term loads

Load duration factor $K_3 = 1.00$

Maximum bending moment $M = 1.069$ kNm

Maximum shear force $V = 1.425$ kN

Maximum support reaction $R = 1.425$ kN

Maximum deflection $\delta = 4.372$ mm

Check bending stress

Bending stress $\sigma_m = 7.500$ N/mm²

Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 8.754$ N/mm²

Applied bending stress $\sigma_{m_max} = M / Z = 4.188$ N/mm²

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress $\tau = 0.710$ N/mm²

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.781$ N/mm²

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h) = 0.244$ N/mm²

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane) $\sigma_{cp1} = 2.400$ N/mm²

Permissible bearing stress $\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 2.640$ N/mm²

Applied bearing stress $\sigma_{c_max} = R / (b \times L_b) = 0.285$ N/mm²

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 9.000$ mm

Bending deflection (based on E_{mean}) $\delta_{bending} = 4.155$ mm

Shear deflection $\delta_{shear} = 0.217$ mm

Total deflection $\delta = \delta_{bending} + \delta_{shear} = 4.372$ mm

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor $K_3 = 1.25$

Maximum bending moment $M = 1.444$ kNm

Maximum shear force $V = 1.925$ kN

Maximum support reaction $R = 1.925$ kN

Maximum deflection $\delta = 5.089$ mm

Check bending stress

Bending stress $\sigma_m = 7.500$ N/mm²

Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 10.942$ N/mm²

Applied bending stress $\sigma_{m_max} = M / Z = 5.657$ N/mm²

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress $\tau = 0.710$ N/mm²

Permissible shear stress $\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.976$ N/mm²

Applied shear stress $\tau_{max} = 3 \times V / (2 \times b \times h) = 0.330$ N/mm²

Project New House at High Street, Trelewis				Job no. 18300	
Calcs for Timber flat roof joists over garage				Start page no./Revision 40	
Calcs by AS	Calcs date 12/02/2024	Checked by MJ	Checked date 12/02/2024	Approved by MJ	Approved date 12/02/2024

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)

$$\sigma_{cp1} = \mathbf{2.400 \text{ N/mm}^2}$$

Permissible bearing stress

$$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = \mathbf{3.300 \text{ N/mm}^2}$$

Applied bearing stress

$$\sigma_{c_max} = R / (b \times L_b) = \mathbf{0.385 \text{ N/mm}^2}$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = \mathbf{9.000 \text{ mm}}$$

Bending deflection (based on E_{mean})

$$\delta_{bending} = \mathbf{4.796 \text{ mm}}$$

Shear deflection

$$\delta_{shear} = \mathbf{0.293 \text{ mm}}$$

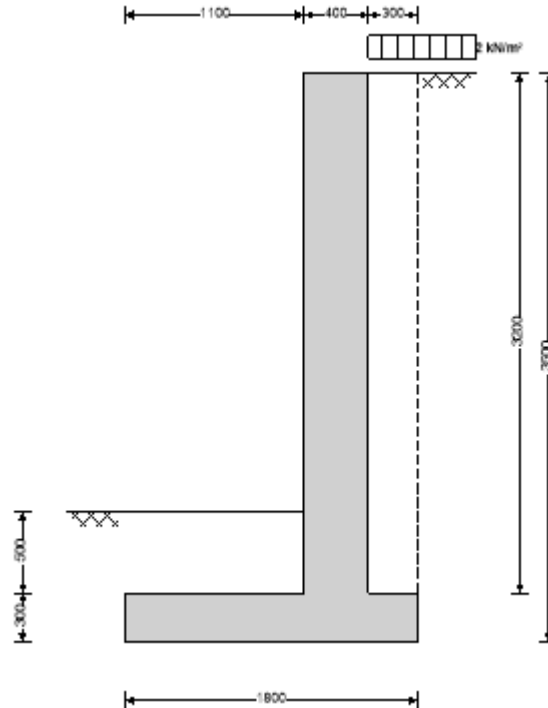
Total deflection

$$\delta = \delta_{bending} + \delta_{shear} = \mathbf{5.089 \text{ mm}}$$

PASS - Actual deflection within permissible limits

RETAINING WALL ANALYSIS (BS 8002:1994)

TEDDS calculation version 1.2.01.08



Wall details

Retaining wall type
Height of retaining wall stem
Thickness of wall stem
Length of toe
Length of heel
Overall length of base
Thickness of base
Depth of downstand
Position of downstand
Thickness of downstand
Height of retaining wall
Depth of cover in front of wall
Depth of unplanned excavation
Height of ground water behind wall
Height of saturated fill above base
Density of wall construction
Density of base construction
Angle of rear face of wall
Angle of soil surface behind wall
Effective height at virtual back of wall

Unpropped cantilever

$h_{\text{stem}} = 3200 \text{ mm}$
 $t_{\text{wall}} = 400 \text{ mm}$
 $l_{\text{toe}} = 1100 \text{ mm}$
 $l_{\text{heel}} = 300 \text{ mm}$
 $l_{\text{base}} = l_{\text{toe}} + l_{\text{heel}} + t_{\text{wall}} = 1800 \text{ mm}$
 $t_{\text{base}} = 300 \text{ mm}$
 $d_{\text{ds}} = 0 \text{ mm}$
 $l_{\text{ds}} = 1150 \text{ mm}$
 $t_{\text{ds}} = 300 \text{ mm}$
 $h_{\text{wall}} = h_{\text{stem}} + t_{\text{base}} + d_{\text{ds}} = 3500 \text{ mm}$
 $d_{\text{cover}} = 500 \text{ mm}$
 $d_{\text{exc}} = 0 \text{ mm}$
 $h_{\text{water}} = 0 \text{ mm}$
 $h_{\text{sat}} = \max(h_{\text{water}} - t_{\text{base}} - d_{\text{ds}}, 0 \text{ mm}) = 0 \text{ mm}$
 $\gamma_{\text{wall}} = 23.6 \text{ kN/m}^3$
 $\gamma_{\text{base}} = 23.6 \text{ kN/m}^3$
 $\alpha = 90.0 \text{ deg}$
 $\beta = 0.0 \text{ deg}$
 $h_{\text{eff}} = h_{\text{wall}} + l_{\text{heel}} \times \tan(\beta) = 3500 \text{ mm}$

Retained material details

Mobilisation factor
Moist density of retained material

$M = 1.5$
 $\gamma_m = 18.0 \text{ kN/m}^3$

Saturated density of retained material

$\gamma_s = 21.0 \text{ kN/m}^3$

Design shear strength

$\phi' = 24.2 \text{ deg}$

Angle of wall friction

$\delta = 18.6 \text{ deg}$

Base material details

Firm clay

Moist density

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

Design shear strength

$\phi'_b = 24.2 \text{ deg}$

Design base friction

$\delta_b = 18.6 \text{ deg}$

Allowable bearing pressure

$P_{bearing} = 100 \text{ kN/m}^2$

Using Coulomb theory

Active pressure coefficient for retained material

$$K_a = \sin(\alpha + \phi')^2 / (\sin(\alpha)^2 \times \sin(\alpha - \delta) \times [1 + \sqrt{(\sin(\phi' + \delta) \times \sin(\phi' - \beta) / (\sin(\alpha - \delta) \times \sin(\alpha + \beta)))^2}] = 0.369$$

Passive pressure coefficient for base material

$$K_p = \sin(90 - \phi'_b)^2 / (\sin(90 - \delta_b) \times [1 - \sqrt{(\sin(\phi'_b + \delta_b) \times \sin(\phi'_b) / (\sin(90 + \delta_b)))^2}] = 4.187$$

At-rest pressure

At-rest pressure for retained material

$K_0 = 1 - \sin(\phi') = 0.590$

Loading details

Surcharge load on plan

Surcharge = **1.5 kN/m²**

Applied vertical dead load on wall

$W_{dead} = 0.0 \text{ kN/m}$

Applied vertical live load on wall

$W_{live} = 0.0 \text{ kN/m}$

Position of applied vertical load on wall

$l_{load} = 0 \text{ mm}$

Applied horizontal dead load on wall

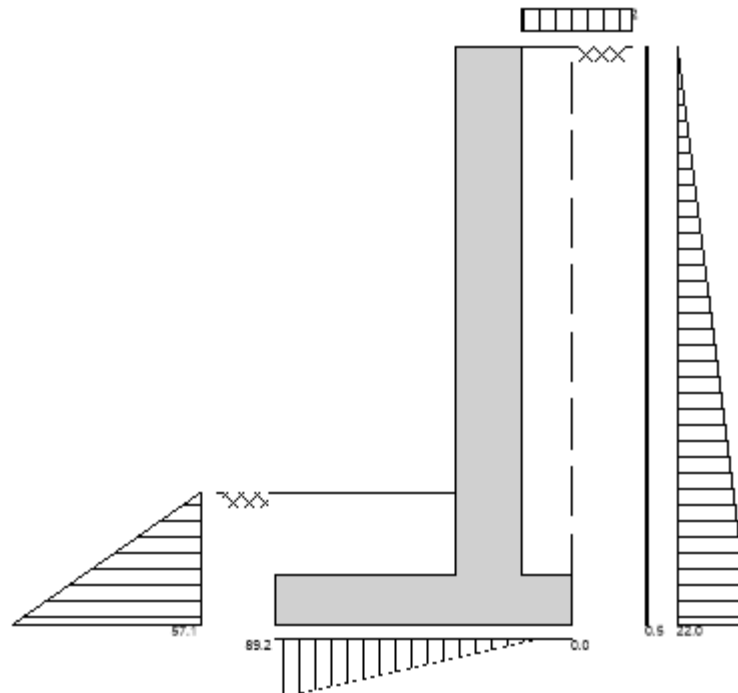
$F_{dead} = 0.0 \text{ kN/m}$

Applied horizontal live load on wall

$F_{live} = 0.0 \text{ kN/m}$

Height of applied horizontal load on wall

$h_{load} = 0 \text{ mm}$



Loads shown in kN/m, pressures shown in kN/m²

Vertical forces on wall

Wall stem

$$W_{wall} = h_{stem} \times t_{wall} \times \gamma_{wall} = \mathbf{30.2 \text{ kN/m}}$$

Wall base

$$W_{base} = l_{base} \times t_{base} \times \gamma_{base} = \mathbf{12.7 \text{ kN/m}}$$

Surcharge

$$W_{sur} = \text{Surcharge} \times l_{heel} = \mathbf{0.5 \text{ kN/m}}$$

Moist backfill to top of wall

$$W_{m_w} = l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m = \mathbf{17.3 \text{ kN/m}}$$

Soil in front of wall

$$W_p = l_{toe} \times d_{cover} \times \gamma_{mb} = \mathbf{9.9 \text{ kN/m}}$$

Total vertical load

$$W_{total} = W_{wall} + W_{base} + W_{sur} + W_{m_w} + W_p = \mathbf{70.6 \text{ kN/m}}$$

Horizontal forces on wall

Surcharge

$$F_{sur} = K_a \times \cos(90 - \alpha + \delta) \times \text{Surcharge} \times h_{eff} = \mathbf{1.8 \text{ kN/m}}$$

Moist backfill above water table

$$F_{m_a} = 0.5 \times K_a \times \cos(90 - \alpha + \delta) \times \gamma_m \times (h_{eff} - h_{water})^2 = \mathbf{38.6 \text{ kN/m}}$$

Total horizontal load

$$F_{total} = F_{sur} + F_{m_a} = \mathbf{40.4 \text{ kN/m}}$$

Calculate stability against sliding

Passive resistance of soil in front of wall

$$F_p = 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} = \mathbf{22.9 \text{ kN/m}}$$

Resistance to sliding

$$F_{res} = F_p + (W_{total} - W_{sur} - W_p) \times \tan(\delta_b) = \mathbf{43.1 \text{ kN/m}}$$

PASS - Resistance force is greater than sliding force

Overturning moments

Surcharge

$$M_{sur} = F_{sur} \times (h_{eff} - 2 \times d_{ds}) / 2 = \mathbf{3.2 \text{ kNm/m}}$$

Moist backfill above water table

$$M_{m_a} = F_{m_a} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3 = \mathbf{45 \text{ kNm/m}}$$

Total overturning moment

$$M_{ot} = M_{sur} + M_{m_a} = \mathbf{48.2 \text{ kNm/m}}$$

Restoring moments

Wall stem

$$M_{wall} = W_{wall} \times (l_{toe} + t_{wall} / 2) = \mathbf{39.3 \text{ kNm/m}}$$

Wall base

$$M_{base} = W_{base} \times l_{base} / 2 = \mathbf{11.5 \text{ kNm/m}}$$

Moist backfill

$$M_{m_r} = (W_{m_w} \times (l_{base} - l_{heel} / 2) + W_{m_s} \times (l_{base} - l_{heel} / 3)) = \mathbf{28.5 \text{ kNm/m}}$$

Total restoring moment

$$M_{rest} = M_{wall} + M_{base} + M_{m_r} = \mathbf{79.3 \text{ kNm/m}}$$

Check stability against overturning

Total overturning moment

$$M_{ot} = \mathbf{48.2 \text{ kNm/m}}$$

Total restoring moment

$$M_{rest} = \mathbf{79.3 \text{ kNm/m}}$$

PASS - Restoring moment is greater than overturning moment

Check bearing pressure

Surcharge

$$M_{sur_r} = W_{sur} \times (l_{base} - l_{heel} / 2) = \mathbf{0.7 \text{ kNm/m}}$$

Soil in front of wall

$$M_{p_r} = W_p \times l_{toe} / 2 = \mathbf{5.4 \text{ kNm/m}}$$

Total moment for bearing

$$M_{total} = M_{rest} - M_{ot} + M_{sur_r} + M_{p_r} = \mathbf{37.2 \text{ kNm/m}}$$

Total vertical reaction

$$R = W_{total} = \mathbf{70.6 \text{ kN/m}}$$

Distance to reaction

$$X_{bar} = M_{total} / R = \mathbf{527 \text{ mm}}$$

Eccentricity of reaction

$$e = \text{abs}((l_{base} / 2) - X_{bar}) = \mathbf{373 \text{ mm}}$$

Reaction acts outside middle third of base

Bearing pressure at toe

$$p_{toe} = R / (1.5 \times X_{bar}) = \mathbf{89.2 \text{ kN/m}^2}$$

Bearing pressure at heel

$$p_{heel} = 0 \text{ kN/m}^2 = \mathbf{0 \text{ kN/m}^2}$$

PASS - Maximum bearing pressure is less than allowable bearing pressure

RETAINING WALL DESIGN (BS 8002:1994)

TEDDS calculation version 1.2.01.08

Ultimate limit state load factors

Dead load factor $\gamma_{f,d} = 1.4$

Live load factor $\gamma_{f,l} = 1.6$

Earth and water pressure factor $\gamma_{f,e} = 1.4$

Factored vertical forces on wall

Wall stem $W_{wall,f} = \gamma_{f,d} \times h_{stem} \times t_{wall} \times \gamma_{wall} = 42.3 \text{ kN/m}$

Wall base $W_{base,f} = \gamma_{f,d} \times l_{base} \times t_{base} \times \gamma_{base} = 17.8 \text{ kN/m}$

Surcharge $W_{sur,f} = \gamma_{f,l} \times \text{Surcharge} \times l_{heel} = 0.7 \text{ kN/m}$

Moist backfill to top of wall $W_{m,w,f} = \gamma_{f,d} \times l_{heel} \times (h_{stem} - h_{sat}) \times \gamma_m = 24.2 \text{ kN/m}$

Soil in front of wall $W_{p,f} = \gamma_{f,d} \times l_{toe} \times d_{cover} \times \gamma_{mb} = 13.9 \text{ kN/m}$

Total vertical load $W_{total,f} = W_{wall,f} + W_{base,f} + W_{sur,f} + W_{m,w,f} + W_{p,f} = 98.9 \text{ kN/m}$

Factored horizontal at-rest forces on wall

Surcharge $F_{sur,f} = \gamma_{f,l} \times K_0 \times \text{Surcharge} \times h_{eff} = 5 \text{ kN/m}$

Moist backfill above water table $F_{m,a,f} = \gamma_{f,e} \times 0.5 \times K_0 \times \gamma_m \times (h_{eff} - h_{water})^2 = 91.1 \text{ kN/m}$

Total horizontal load $F_{total,f} = F_{sur,f} + F_{m,a,f} = 96 \text{ kN/m}$

Passive resistance of soil in front of wall $F_{p,f} = \gamma_{f,e} \times 0.5 \times K_p \times \cos(\delta_b) \times (d_{cover} + t_{base} + d_{ds} - d_{exc})^2 \times \gamma_{mb} = 32 \text{ kN/m}$

Factored overturning moments

Surcharge $M_{sur,f} = F_{sur,f} \times (h_{eff} - 2 \times d_{ds}) / 2 = 8.7 \text{ kNm/m}$

Moist backfill above water table $M_{m,a,f} = F_{m,a,f} \times (h_{eff} + 2 \times h_{water} - 3 \times d_{ds}) / 3 = 106.3 \text{ kNm/m}$

Total overturning moment $M_{ot,f} = M_{sur,f} + M_{m,a,f} = 114.9 \text{ kNm/m}$

Restoring moments

Wall stem $M_{wall,f} = W_{wall,f} \times (l_{toe} + t_{wall} / 2) = 55 \text{ kNm/m}$

Wall base $M_{base,f} = W_{base,f} \times l_{base} / 2 = 16.1 \text{ kNm/m}$

Surcharge $M_{sur,r,f} = W_{sur,f} \times (l_{base} - l_{heel} / 2) = 1.2 \text{ kNm/m}$

Moist backfill $M_{m,r,f} = (W_{m,w,f} \times (l_{base} - l_{heel} / 2) + W_{m,s,f} \times (l_{base} - l_{heel} / 3)) = 39.9 \text{ kNm/m}$

Soil in front of wall $M_{p,r,f} = W_{p,f} \times l_{toe} / 2 = 7.6 \text{ kNm/m}$

Total restoring moment $M_{rest,f} = M_{wall,f} + M_{base,f} + M_{sur,r,f} + M_{m,r,f} + M_{p,r,f} = 119.8 \text{ kNm/m}$

Factored bearing pressure

Total moment for bearing $M_{total,f} = M_{rest,f} - M_{ot,f} = 4.8 \text{ kNm/m}$

Total vertical reaction $R_f = W_{total,f} = 98.9 \text{ kN/m}$

Distance to reaction $X_{bar,f} = M_{total,f} / R_f = 49 \text{ mm}$

Eccentricity of reaction $e_f = \text{abs}((l_{base} / 2) - X_{bar,f}) = 851 \text{ mm}$
Reaction acts outside middle third of base

Bearing pressure at toe $p_{toe,f} = R_f / (1.5 \times X_{bar,f}) = 1349.8 \text{ kN/m}^2$

Bearing pressure at heel $p_{heel,f} = 0 \text{ kN/m}^2 = 0 \text{ kN/m}^2$

Rate of change of base reaction $\text{rate} = p_{toe,f} / (3 \times X_{bar,f}) = 9210.11 \text{ kN/m}^2/\text{m}$

Bearing pressure at stem / toe $p_{stem,toe,f} = \text{max}(p_{toe,f} - (\text{rate} \times l_{toe}), 0 \text{ kN/m}^2) = 0 \text{ kN/m}^2$

Bearing pressure at mid stem $p_{stem,mid,f} = \text{max}(p_{toe,f} - (\text{rate} \times (l_{toe} + t_{wall} / 2)), 0 \text{ kN/m}^2) = 0 \text{ kN/m}^2$

Bearing pressure at stem / heel $p_{stem,heel,f} = \text{max}(p_{toe,f} - (\text{rate} \times (l_{toe} + t_{wall})), 0 \text{ kN/m}^2) = 0 \text{ kN/m}^2$

Design of reinforced concrete retaining wall toe (BS 8002:1994)

Material properties

Characteristic strength of concrete

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

Characteristic strength of reinforcement

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

Base details

Minimum area of reinforcement

$$k = 0.13 \%$$

Cover to reinforcement in toe

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

Calculate shear for toe design

Shear from bearing pressure

$$V_{toe_bear} = 3 \times p_{toe_f} \times X_{bar_f} / 2 = 98.9 \text{ kN/m}$$

Shear from weight of base

$$V_{toe_wt_base} = \gamma_{f_d} \times \gamma_{base} \times l_{toe} \times t_{base} = 10.9 \text{ kN/m}$$

Shear from weight of soil

$$V_{toe_wt_soil} = W_{p_f} - (\gamma_{f_d} \times \gamma_m \times l_{toe} \times d_{exc}) = 13.9 \text{ kN/m}$$

Total shear for toe design

$$V_{toe} = V_{toe_bear} - V_{toe_wt_base} - V_{toe_wt_soil} = 74.1 \text{ kN/m}$$

Calculate moment for toe design

Moment from bearing pressure

$$M_{toe_bear} = 3 \times p_{toe_f} \times X_{bar_f} \times (l_{toe} - X_{bar_f} + t_{wall} / 2) / 2 = 123.7 \text{ kNm/m}$$

Moment from weight of base

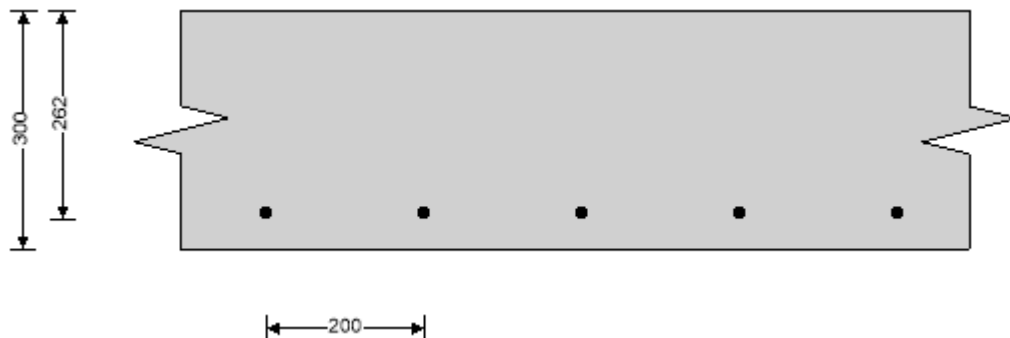
$$M_{toe_wt_base} = (\gamma_{f_d} \times \gamma_{base} \times t_{base} \times (l_{toe} + t_{wall} / 2)^2 / 2) = 8.4 \text{ kNm/m}$$

Moment from weight of soil

$$M_{toe_wt_soil} = (W_{p_f} - (\gamma_{f_d} \times \gamma_m \times l_{toe} \times d_{exc})) \times (l_{toe} + t_{wall}) / 2 = 10.4 \text{ kNm/m}$$

Total moment for toe design

$$M_{toe} = M_{toe_bear} - M_{toe_wt_base} - M_{toe_wt_soil} = 105 \text{ kNm/m}$$



Check toe in bending

Width of toe

$$b = 1000 \text{ mm/m}$$

Depth of reinforcement

$$d_{toe} = t_{base} - C_{toe} - (\phi_{toe} / 2) = 262.0 \text{ mm}$$

Constant

$$K_{toe} = M_{toe} / (b \times d_{toe}^2 \times f_{cu}) = 0.044$$

Compression reinforcement is not required

Lever arm

$$Z_{toe} = \min(0.5 + \sqrt{(0.25 - (\min(K_{toe}, 0.225) / 0.9))}, 0.95) \times d_{toe}$$

$$Z_{toe} = 249 \text{ mm}$$

Area of tension reinforcement required

$$A_{s_toe_des} = M_{toe} / (0.87 \times f_y \times Z_{toe}) = 971 \text{ mm}^2/\text{m}$$

Minimum area of tension reinforcement

$$A_{s_toe_min} = k \times b \times t_{base} = 390 \text{ mm}^2/\text{m}$$

Area of tension reinforcement required

$$A_{s_toe_req} = \text{Max}(A_{s_toe_des}, A_{s_toe_min}) = 971 \text{ mm}^2/\text{m}$$

Reinforcement provided

$$16 \text{ mm dia. bars @ 200 mm centres}$$

Area of reinforcement provided

$$A_{s_toe_prov} = 1005 \text{ mm}^2/\text{m}$$

PASS - Reinforcement provided at the retaining wall toe is adequate

Check shear resistance at toe

Design shear stress

$$v_{toe} = V_{toe} / (b \times d_{toe}) = 0.283 \text{ N/mm}^2$$

Allowable shear stress

$$v_{adm} = \min(0.8 \times \sqrt{f_{cu}} / 1 \text{ N/mm}^2, 5) \times 1 \text{ N/mm}^2 = 4.733 \text{ N/mm}^2$$

PASS - Design shear stress is less than maximum shear stress

From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress

$$v_{c_toe} = 0.571 \text{ N/mm}^2$$

$V_{toe} < V_{c_toe}$ - No shear reinforcement required

Design of reinforced concrete retaining wall heel (BS 8002:1994)

Material properties

Characteristic strength of concrete $f_{cu} = 35 \text{ N/mm}^2$

Characteristic strength of reinforcement $f_y = 500 \text{ N/mm}^2$

Base details

Minimum area of reinforcement $k = 0.13 \%$

Cover to reinforcement in heel $C_{heel} = 30 \text{ mm}$

Calculate shear for heel design

Shear from weight of base $V_{heel_wt_base} = \gamma_{fd} \times \gamma_{base} \times l_{heel} \times t_{base} = 3 \text{ kN/m}$

Shear from weight of moist backfill $V_{heel_wt_m} = W_{m_w_f} = 24.2 \text{ kN/m}$

Shear from surcharge $V_{heel_sur} = W_{sur_f} = 0.7 \text{ kN/m}$

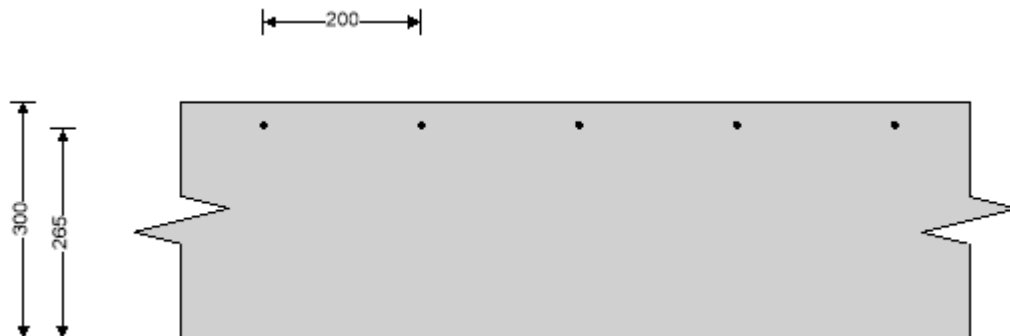
Total shear for heel design $V_{heel} = V_{heel_wt_base} + V_{heel_wt_m} + V_{heel_sur} = 27.9 \text{ kN/m}$

Calculate moment for heel design

Moment from weight of base $M_{heel_wt_base} = (\gamma_{fd} \times \gamma_{base} \times t_{base} \times (l_{heel} + t_{wall} / 2)^2 / 2) = 1.2 \text{ kNm/m}$

Moment from weight of moist backfill $M_{heel_wt_m} = W_{m_w_f} \times (l_{heel} + t_{wall}) / 2 = 8.5 \text{ kNm/m}$

Moment from surcharge $M_{heel_sur} = W_{sur_f} \times (l_{heel} + t_{wall}) / 2 = 0.3 \text{ kNm/m}$

Total moment for heel design $M_{heel} = M_{heel_wt_base} + M_{heel_wt_m} + M_{heel_sur} = 10 \text{ kNm/m}$


Check heel in bending

Width of heel $b = 1000 \text{ mm/m}$

Depth of reinforcement $d_{heel} = t_{base} - C_{heel} - (\phi_{heel} / 2) = 265.0 \text{ mm}$

Constant $K_{heel} = M_{heel} / (b \times d_{heel}^2 \times f_{cu}) = 0.004$
Compression reinforcement is not required

Lever arm $Z_{heel} = \min(0.5 + \sqrt{(0.25 - (\min(K_{heel}, 0.225) / 0.9))}, 0.95) \times d_{heel}$
 $Z_{heel} = 252 \text{ mm}$

Area of tension reinforcement required $A_{s_heel_des} = M_{heel} / (0.87 \times f_y \times Z_{heel}) = 91 \text{ mm}^2/\text{m}$

Minimum area of tension reinforcement $A_{s_heel_min} = k \times b \times t_{base} = 390 \text{ mm}^2/\text{m}$

Area of tension reinforcement required $A_{s_heel_req} = \text{Max}(A_{s_heel_des}, A_{s_heel_min}) = 390 \text{ mm}^2/\text{m}$

Reinforcement provided

A393 mesh

Area of reinforcement provided

 $A_{s_heel_prov} = 393 \text{ mm}^2/\text{m}$
PASS - Reinforcement provided at the retaining wall heel is adequate

Check shear resistance at heel

Design shear stress $V_{heel} = V_{heel} / (b \times d_{heel}) = 0.105 \text{ N/mm}^2$

Allowable shear stress $V_{adm} = \min(0.8 \times \sqrt{f_{cu}} / 1 \text{ N/mm}^2, 5) \times 1 \text{ N/mm}^2 = 4.733 \text{ N/mm}^2$
PASS - Design shear stress is less than maximum shear stress

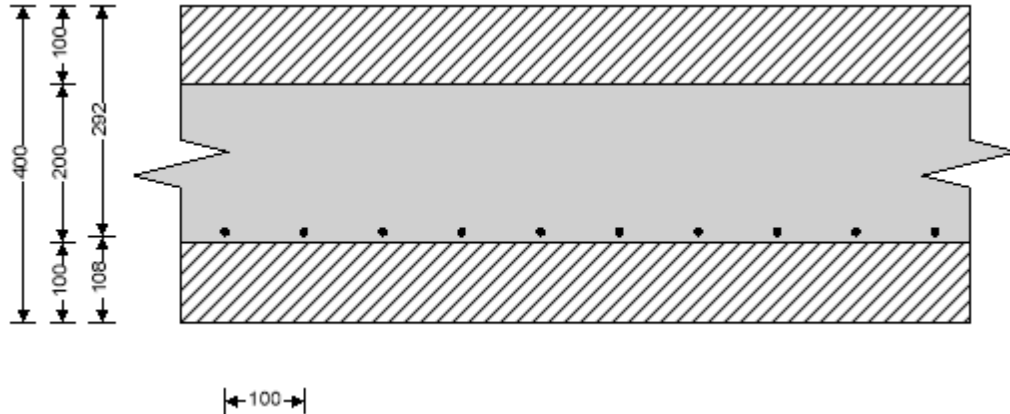
From BS8110:Part 1:1997 – Table 3.8

Design concrete shear stress

$$V_{c_heel} = 0.415 \text{ N/mm}^2$$

$V_{heel} < V_{c_heel}$ - No shear reinforcement required

Design of cavity reinforced masonry retaining wall stem - BS5628-2:2000



Wall details

Thickness of outer leaf of wall

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

Thickness of inner leaf of wall

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

Thickness of reinforced cavity

$$t_{cavity} = t_{wall} - t_{outer} - t_{inner} = 200 \text{ mm}$$

Depth of stem reinforcement

$$d_{stem} = 292 \text{ mm}$$

Masonry details

Masonry type

Aggregate concrete blocks no voids

Compressive strength of units

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

Mortar designation

(ii)

Category of manufacturing control of units

Category I

Partial safety factor for material strength

$$\gamma_{mm} = 2.0$$

Characteristic compressive strength of masonry

Least horizontal dimension of masonry units

$$b_{unit} = 100.0 \text{ mm}$$

Height of masonry units

$$h_{unit} = 215.0 \text{ mm}$$

Ratio of height to least horizontal dimension

$$\text{ratio} = h_{unit} / b_{unit} = 2.2$$

From BS5628:2 Table 3d, mortar ii

Characteristic compressive strength

$$f_k = 6.4 \text{ N/mm}^2$$

Factored horizontal at-rest forces on stem

Surcharge

$$F_{s_sur_f} = \gamma_{f_l} \times K_0 \times \text{Surcharge} \times (h_{eff} - t_{base} - d_{ds}) = 4.5 \text{ kN/m}$$

Moist backfill above water table

$$F_{s_m_a_f} = 0.5 \times \gamma_{f_e} \times K_0 \times \gamma_m \times (h_{eff} - t_{base} - d_{ds} - h_{sat})^2 = 76.1 \text{ kN/m}$$

Calculate shear for stem design

Shear at base of stem

$$V_{stem} = F_{s_sur_f} + F_{s_m_a_f} = 80.7 \text{ kN/m}$$

Calculate moment for stem design

Surcharge

$$M_{s_sur} = F_{s_sur_f} \times (h_{stem} + t_{base}) / 2 = 7.9 \text{ kNm/m}$$

Moist backfill above water table

$$M_{s_m_a} = F_{s_m_a_f} \times (2 \times h_{sat} + h_{eff} - d_{ds} + t_{base} / 2) / 3 = 92.6 \text{ kNm/m}$$

Total moment for stem design

$$M_{stem} = M_{s_sur} + M_{s_m_a} = 100.6 \text{ kNm/m}$$

Check maximum design moment for wall stem

Width of wall

$$b = 1000 \text{ mm/m}$$

Maximum design moment

$$M_{d_stem} = 0.4 \times f_k \times b \times d_{stem}^2 / \gamma_{mm} = 109.1 \text{ kNm/m}$$

PASS - Applied moment is less than maximum design moment

Check wall stem in bending

Moment of resistance factor

$$Q = M_{stem} / d_{stem}^2 = \mathbf{1.179 \text{ N/mm}^2}$$

$$Q = 2 \times c \times (1 - c) \times f_k / \gamma_{mm}$$

Lever arm factor

$$c = \mathbf{0.756}$$

Lever arm

$$Z_{stem} = \min(0.95, c) \times d_{stem} = \mathbf{220.9 \text{ mm}}$$

Area of tension reinforcement required

$$A_{s_stem_des} = M_{stem} \times \gamma_{ms} / (f_y \times Z_{stem}) = \mathbf{1047 \text{ mm}^2/\text{m}}$$

Minimum area of tension reinforcement

$$A_{s_stem_min} = k \times b \times t_{wall} = \mathbf{520 \text{ mm}^2/\text{m}}$$

Area of tension reinforcement required

$$A_{s_stem_req} = \text{Max}(A_{s_stem_des}, A_{s_stem_min}) = \mathbf{1047 \text{ mm}^2/\text{m}}$$

Reinforcement provided

$$\mathbf{12 \text{ mm dia. bars @ } 100 \text{ mm centres}}$$

Area of reinforcement provided

$$A_{s_stem_prov} = \pi \times \phi_{stem}^2 / (4 \times S_{stem}) = \mathbf{1131 \text{ mm}^2/\text{m}}$$

PASS - Reinforcement provided at the retaining wall stem is adequate

Check shear resistance at wall stem

Design shear stress

$$V_{stem} = V_{stem} / (b \times d_{stem}) = \mathbf{0.276 \text{ N/mm}^2}$$

Basic characteristic shear strength of masonry

$$f_{vbas} = \min[0.35 + (17.5 \times A_{s_stem_prov} / (b \times d_{stem})), 0.7] \times 1 \text{ N/mm}^2$$

$$f_{vbas} = \mathbf{0.418 \text{ N/mm}^2}$$

Shear span

$$a = M_{stem} / V_{stem} = \mathbf{1246.6 \text{ mm}}$$

Characteristic shear strength of masonry

$$f_v = \text{Min}(f_{vbas} \times \max(2.5 - 0.25 \times (a / d_{stem}), 1), 1.75 \text{ N/mm}^2)$$

$$f_v = \mathbf{0.599 \text{ N/mm}^2}$$

Allowable shear stress

$$V_{adm} = f_v / \gamma_{mv} = \mathbf{0.299 \text{ N/mm}^2}$$

PASS - Design shear stress is less than maximum shear stress

Check limiting dimensions

Limiting span/effective depth ratio

$$\text{ratio}_{\max} = \mathbf{18.00}$$

Actual span/effective depth ratio

$$\text{ratio}_{\text{act}} = (h_{stem} + d_{stem} / 2) / d_{stem} = \mathbf{11.46}$$

PASS - Span to depth ratio is acceptable

Axial load check

Factored axial load on wall

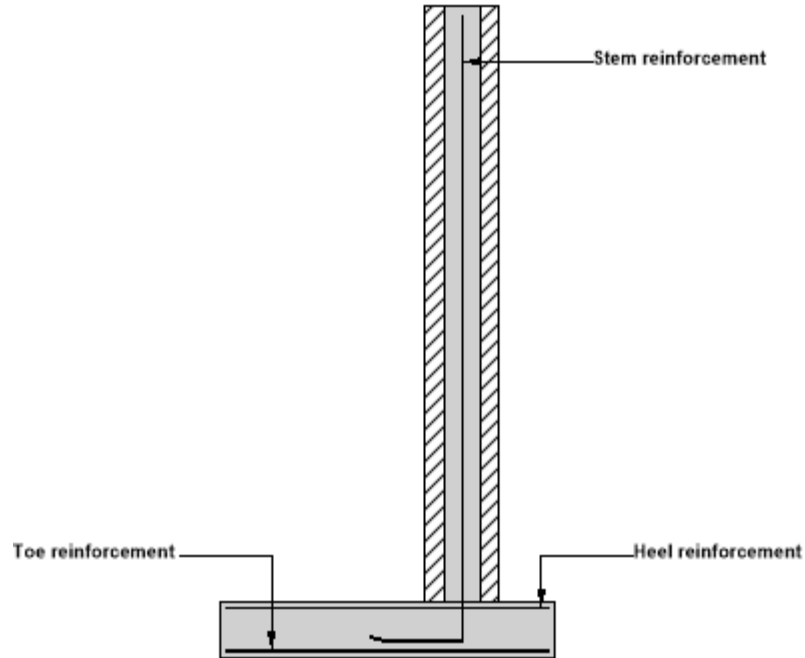
$$N_{wall} = ([t_{wall} \times h_{stem} \times \gamma_{wall} + W_{dead}] \times \gamma_{f_d}) + (W_{live} \times \gamma_{f_l}) = \mathbf{42.3 \text{ kN/m}}$$

Limiting axial load

$$N_{limit} = 0.1 \times f_k \times t_{wall} = \mathbf{256.0 \text{ kN/m}}$$

Applied axial load may be ignored - calculations valid

Indicative retaining wall reinforcement diagram



Toe bars - 16 mm dia.@ 200 mm centres - (1005 mm²/m)

Heel mesh - A393 - (393 mm²/m)

Stem bars - 12 mm dia.@ 100 mm centres - (1131 mm²/m)

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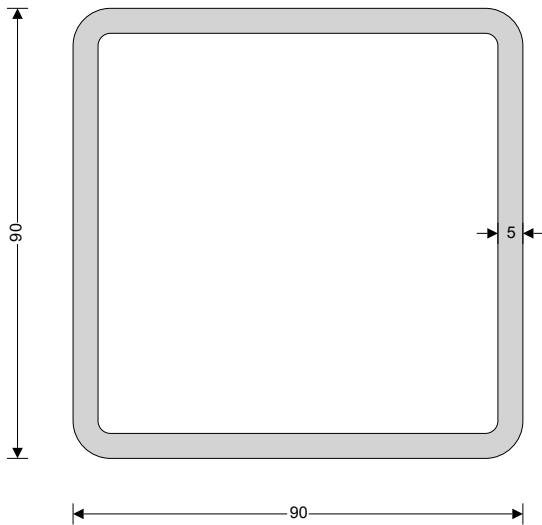
STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.07

Section details

Section type	SHS 90x90x5.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))
Steel grade	S275
From table 9: Design strength p_y	
Thickness of element	$t = 5.0 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Distance between major axis restraints	$L_x = 3000 \text{ mm}$
Distance between minor axis restraints	$L_y = 3000 \text{ mm}$

Effective length factors

Effective length factor in major axis	$K_x = 1.00$
Effective length factor in minor axis	$K_y = 1.00$
Effective length factor for lateral-torsional buckling	$K_{LT} = 1.00$

Classification of cross sections - Section 3.5

$\epsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 1.00$

Web - major axis - Table 12

Depth of section	$d = D - 3 \times t = 75 \text{ mm}$
Stress ratios	$r1 = \min(F_c / (2 \times d \times t \times p_{yw}), 1) = 0.145$
	$r2 = F_c / (A \times p_{yw}) = 0.065$
	$d / t = 15.0 \times \epsilon \leq \max(64 \times \epsilon / (1 + r1), 40 \times \epsilon)$ Class 1 plastic

Flange - major axis - Table 12

Width of section	$b = B - 3 \times t = 75 \text{ mm}$
	$b / t = 15.0 \times \epsilon \leq 40 \times \epsilon$ Class 3 semi-compact

Web - minor axis - Table 12

Depth of section	$d = B - 3 \times t = 75 \text{ mm}$
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Stress ratios

$$r1 = \min(F_c / (2 \times d \times t \times p_{yw}), 1) = \mathbf{0.145}$$

$$r2 = F_c / (A \times p_{yw}) = \mathbf{0.065}$$

$$d / t = 15.0 \times \varepsilon \leq \max(64 \times \varepsilon / (1 + r1), 40 \times \varepsilon) \quad \text{Class 1 plastic}$$

Flange - minor axis - Table 12

Width of section

$$b = D - 3 \times t = \mathbf{75 \text{ mm}}$$

$$b / t = 15.0 \times \varepsilon \leq 40 \times \varepsilon$$

Class 3 semi-compact

Section is class 3 semi-compact

Moment capacity major (x-x) axis - Section 4.2.5

Design bending moment

$$M_x = \mathbf{2 \text{ kNm}}$$

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange

$$\beta_{2f} = \min(32 \times \varepsilon, 62 \times \varepsilon - 0.5 \times d / t) = \mathbf{32}$$

Limiting value for class 3 semi-compact flange

$$\beta_{3f} = 40 \times \varepsilon = \mathbf{40}$$

Limiting value for class 2 compact web

$$\beta_{2w} = \max(80 \times \varepsilon / (1 + r1), 40 \times \varepsilon) = \mathbf{69.841}$$

Limiting value for class 3 semi-compact web

$$\beta_{3w} = \max(120 \times \varepsilon / (1 + 2 \times r2), 40 \times \varepsilon) = \mathbf{106.157}$$

Effective plastic modulus - cl.3.5.6.3

$$S_{eff} = \min(Z + (S - Z) \times \min([(\beta_{3w} / (d / t) - 1) / (\beta_{3w} / \beta_{2w} - 1)], [(\beta_{3f} / (b / t) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S) = \mathbf{52992 \text{ mm}^3}$$

Moment capacity low shear - cl.4.2.5.2

$$M_{cx} = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z) = \mathbf{14.6 \text{ kNm}}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 1.0 \times L_y = \mathbf{3000 \text{ mm}}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = \mathbf{86.860}$$

Equivalent slenderness - Annex B.2.6.1

Torsion constant

$$J = \mathbf{3155046 \text{ mm}^4}$$

$$\gamma_b = (1 - I_{yy} / I_{xx}) \times (1 - J / (2.6 \times I_{xx})) = \mathbf{0.000}$$

$$\phi_b = [S_{xx}^2 \times \gamma_b / (A \times J)]^{0.5} = \mathbf{0.000}$$

Ratio - cl.4.3.6.9

$$\beta_W = S_{eff} / S_{xx} = \mathbf{1.000}$$

Equivalent slenderness

$$\lambda_{LT} = 2.25 \times \sqrt{\phi_b \times \lambda \times \beta_W} = \mathbf{0.000}$$

Limiting slenderness - Annex B.2.2

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{34.310}$$

$\lambda_{LT} < \lambda_{L0}$ - **No allowance need be made for lateral-torsional buckling**

Buckling resistance moment - Section 4.3.6.4

Bending strength

$$p_b = p_y = \mathbf{275 \text{ N/mm}^2}$$

Buckling resistance moment

$$M_b = p_b \times S_{eff} = \mathbf{14.6 \text{ kNm}}$$

PASS - Moment capacity exceeds design bending moment

Moment capacity minor (y-y) axis - Section 4.2.5

Design bending moment

$$M_y = \mathbf{1 \text{ kNm}}$$

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange

$$\beta_{2f} = \min(32 \times \varepsilon, 62 \times \varepsilon - 0.5 \times d / t) = \mathbf{32}$$

Limiting value for class 3 semi-compact flange

$$\beta_{3f} = 40 \times \varepsilon = \mathbf{40}$$

Limiting value for class 2 compact web

$$\beta_{2w} = \max(80 \times \varepsilon / (1 + r1), 40 \times \varepsilon) = \mathbf{69.841}$$

Limiting value for class 3 semi-compact web

$$\beta_{3w} = \max(120 \times \varepsilon / (1 + 2 \times r2), 40 \times \varepsilon) = \mathbf{106.157}$$

Effective plastic modulus - cl.3.5.6.3

$$S_{eff} = \min(Z + (S - Z) \times \min([(\beta_{3w} / (d / t) - 1) / (\beta_{3w} / \beta_{2w} - 1)], [(\beta_{3f} / (b / t) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S) = \mathbf{52992 \text{ mm}^3}$$

Moment capacity low shear - cl.4.2.5.2

$$M_{cy} = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z) = \mathbf{14.6 \text{ kNm}}$$

PASS - Moment capacity exceeds design bending moment

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Compression members - Section 4.7

Design compression force $F_c = 30 \text{ kN}$

Effective length for major (x-x) axis buckling - Section 4.7.3

Effective length for buckling $L_{Ex} = L_x \times K_x = 3000 \text{ mm}$

Slenderness ratio - cl.4.7.2 $\lambda_{xx} = L_{Ex} / r_{xx} = 86.860$

Compressive strength - Section 4.7.5

Limiting slenderness $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5} = 17.155$

Strut curve - Table 23 a

Robertson constant $\alpha_x = 2.0$

Perry factor $\eta_x = \alpha_x \times (\lambda_x - \lambda_0) / 1000 = 0.139$

Euler stress $p_{Ex} = \pi^2 \times E / \lambda_x^2 = 268.2 \text{ N/mm}^2$

$\phi_x = (p_y + (\eta_x + 1) \times p_{Ex}) / 2 = 290.3 \text{ N/mm}^2$

Compressive strength - Annex C.1 $p_{cx} = p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5}) = 187.7 \text{ N/mm}^2$

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 $P_{cx} = A \times p_{cx} = 314.1 \text{ kN}$

PASS - Compression resistance exceeds design compression force

Effective length for minor (y-y) axis buckling - Section 4.7.3

Effective length for buckling $L_{Ey} = L_y \times K_y = 3000 \text{ mm}$

Slenderness ratio - cl.4.7.2 $\lambda_{yy} = L_{Ey} / r_{yy} = 86.860$

Compressive strength - Section 4.7.5

Limiting slenderness $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5} = 17.155$

Strut curve - Table 23 a

Robertson constant $\alpha_y = 2.0$

Perry factor $\eta_y = \alpha_y \times (\lambda_y - \lambda_0) / 1000 = 0.139$

Euler stress $p_{Ey} = \pi^2 \times E / \lambda_y^2 = 268.2 \text{ N/mm}^2$

$\phi_y = (p_y + (\eta_y + 1) \times p_{Ey}) / 2 = 290.3 \text{ N/mm}^2$

Compressive strength - Annex C.1 $p_{cy} = p_{Ey} \times p_y / (\phi_y + (\phi_y^2 - p_{Ey} \times p_y)^{0.5}) = 187.7 \text{ N/mm}^2$

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 $P_{cy} = A \times p_{cy} = 314.1 \text{ kN}$

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comb.compression & bending check - cl.4.8.3.2 $F_c / (A \times p_y) + M_x / M_{cx} + M_y / M_{cy} = 0.271$

PASS - Combined bending and compression check is satisfied

Member buckling resistance - Section 4.8.3.3

Max major axis moment governing $M_{LT} = M_x = 2.00 \text{ kNm}$

Equivalent uniform moment factor for major axis flexural buckling

$m_x = 1.000$

$m_y = 1.000$

Buckling resistance checks - cl.4.8.3.3.3 $F_c / P_{cx} + m_x \times M_x / M_{cx} \times (1 + 0.5 \times F_c / P_{cx}) + 0.5 \times m_{yx} \times M_y / M_{cy} = 0.274$

$F_c / P_{cy} + 0.5 \times m_{LT} \times M_{LT} / M_{cx} + m_y \times M_y / M_{cy} \times (1 + 0.5 \times F_c / P_{cy}) = 0.236$

Interactive buckling $m_x \times M_x \times (1 + 0.5 \times (F_c / P_{cx})) / (M_{cx} \times (1 - F_c / P_{cx})) + m_y \times M_y \times (1 + 0.5 \times (F_c / P_{cy})) / (M_{cy} \times (1 - F_c / P_{cy})) = 0.238$

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PASS - Member buckling resistance checks are satisfied