

Title: Structural Calculations

**Project: Capel Y Glyn, Heol Y Glyn.
 SA11 5BB**

Client: Arran Booth

Document No: 001
Project No: 18529
Prepared By: KA
Checked By: NF

Revision: -
Date: 22/11/2023

DESIGN STATEMENT

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

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

The design is for 2no. steel beams supporting first floor, 1no. ridge beam in rear building, 2no. steel frame in rear building, New Floor joists, New Roof rafter in rear building and masonry bearing (worst-case).

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 100kPa, to be confirmed on site.

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

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

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.

CDM Comments:

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

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	CAPEL Y GLYN, HEOL Y GLYN. SA11 5BB				18529	
	Section				Sheet no./rev.	
	Design loadings				03	
	Calc. by	Date	Chk'd by	Date	App'd by	Date
	KA	22/11/2023	MJ	22/11/2023	MJ	22/11/2023

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

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

		0.90 kN/m ² /cos(35) = 1.10kN/m ² Total Dead Loading

Imposed Loading:		
Snow	=	0.60 kN/m ² Total Live Loading

2. Roof Flat

Dead Loading:		
Waterproof Single Ply Membrane	=	0.10 kN/m ²
Plyboard	=	0.15 kN/m ²
Joists	=	0.15 kN/m ²
Insulation	=	0.05 kN/m ²
Plasterboard & Skim	=	0.15 kN/m ²

		0.60 kN/m ² Total Dead Loading

Imposed Loading:		
Snow	=	0.60 kN/m ² Total Live Loading

3. Walls

Dead Loading:		
Render	=	0.15 kN/m ²
200mm masonry	=	4.30kN/m ²
Plaster	=	0.15 kN/m ²

		4.60 kN/m ² Total Dead Loading

Stud		
Dead Loading:		
Plasterboard	=	0.15 kN/m ²
Render/Render Skin	=	0.20kN/m ²
Stud	=	0.06kN/m ²
OSB	=	0.12 kN/m ²

		0.53 kN/m ² Total Dead Loading

4. Attic / First 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

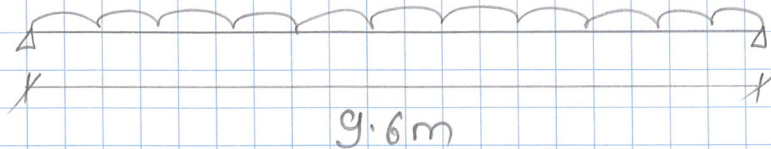
		0.5 kN/m ² Total Dead Loading

Imposed Loading:		
Residential	=	1.50 kN/m ² Total Live Loading



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FIRST FLOOR BEAM -1



Design span = 9.6m

Supporting;

$$\text{Floor Joists} = \frac{9.3\text{m}}{2} = 4.65\text{m wide}$$

Loading;

$$G_k \Rightarrow \text{Floor Joist} = 0.5 \text{ kN/m}^2 \times 4.65\text{m} = 2.33 \text{ kN/m}$$

$$Q_k \Rightarrow \text{Floor Joist} = 1.5 \text{ kN/m}^2 \times 4.65\text{m} = 7 \text{ kN/m}$$

We 356x171x67 UB

$$R_{a, \text{dead}} = 14.3 \text{ kN}$$

$$R_{a, \text{live}} = 33.6 \text{ kN}$$

$$R_{a, \text{max}} = 73.8 \text{ kN}$$



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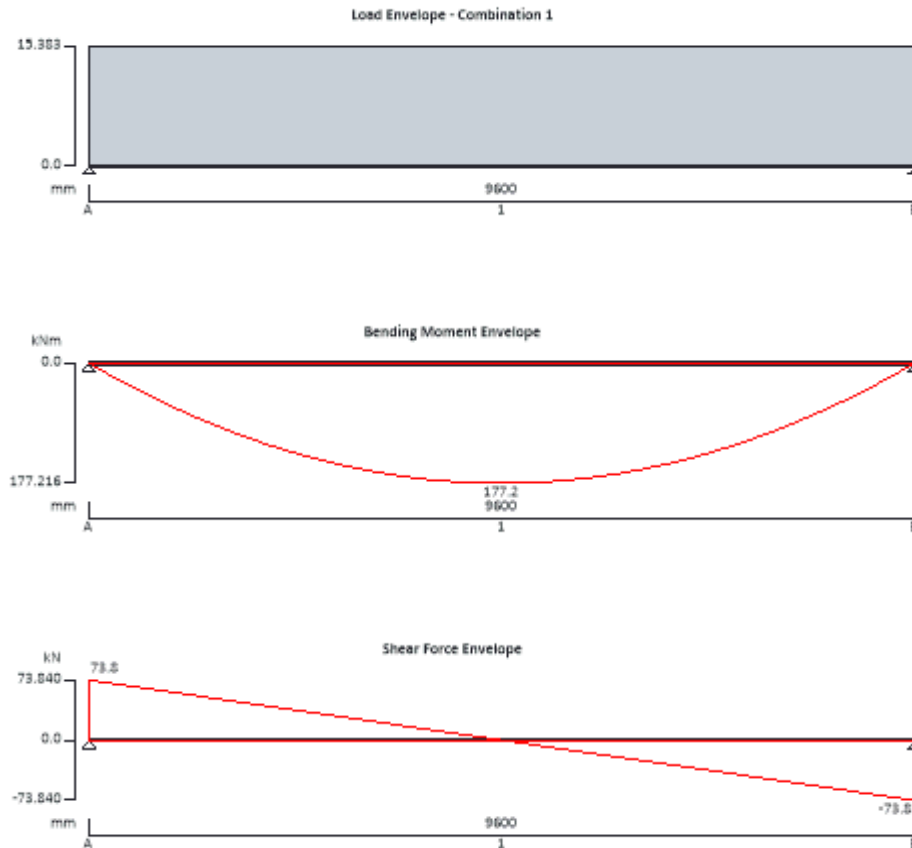
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Calcs for First Floor Beam				Start page no./Revision 5	
Calcs by KA	Calcs date 17/11/2023	Checked by NF	Checked date 17/11/2023	Approved by NF	Approved date 17/11/2023

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 2.33 kN/m
Floor - Imposed full UDL 7 kN/m

Load combinations

Load combination 1

Support A

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

Support B

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



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Analysis results

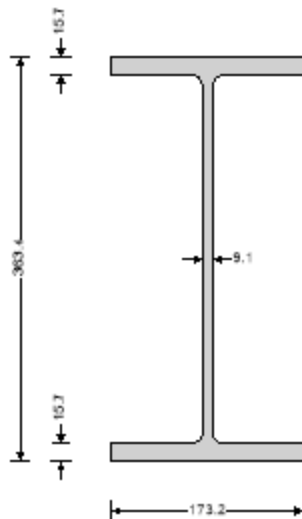
Maximum moment	$M_{max} = 177.2 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 73.8 \text{ kN}$	$V_{min} = -73.8 \text{ kN}$
Deflection	$\delta_{max} = 27.7 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{max}} = 73.8 \text{ kN}$	$R_{A_{min}} = 73.8 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{Dead}} = 14.3 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{Imposed}} = 33.6 \text{ kN}$	
Maximum reaction at support B	$R_{B_{max}} = 73.8 \text{ kN}$	$R_{B_{min}} = 73.8 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{Dead}} = 14.3 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{Imposed}} = 33.6 \text{ kN}$	

Section details

Section type	UKB 356x171x67 (Tata Steel Advance)
Steel grade	S355

From table 9: Design strength p_y

Thickness of element	$\max(T, t) = 15.7 \text{ mm}$
Design strength	$p_y = 355 \text{ N/mm}^2$
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

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 0.88$$

Internal compression parts - Table 11

Depth of section	$d = 311.6 \text{ mm}$	
	$d / t = 38.9 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

Outstand flanges - Table 11

Width of section	$b = B / 2 = 86.6 \text{ mm}$
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$$b / T = 6.3 \times \varepsilon \leq 9 \times \varepsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{73.8 \text{ kN}}$$

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

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = \mathbf{3307 \text{ mm}^2}$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = \mathbf{704.4 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{177.2 \text{ kNm}}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{429.9 \text{ kNm}}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 250 = \mathbf{38.4 \text{ mm}}$$

Maximum deflection span 1

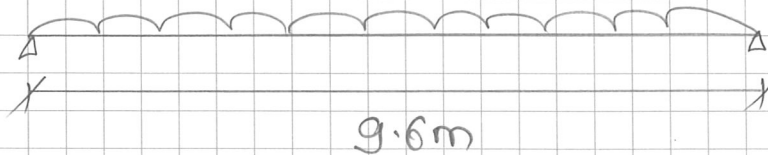
$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{27.685 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit



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FIRST FLOOR BEAM-2



Design Span = 9.6m

Supporting;

Floor Joists = $\frac{4.65m}{2} = 2.3m$ wide

Wall (Stud) = $\frac{2}{2.4m}$ High

Balustrade = 1m High.

Loading

GK $\Rightarrow$ Floor Joists = $0.5 kN/m^2 \times 2.3m = 1.2 kN/m$

Wall (Stud) = $0.53 kN/m^2 \times 2.4m = 1.3 kN/m$

Balustrade = $1 kN/m^2 \times 1m = 1 kN/m$

Δ K $\Rightarrow$ Floor Joists = $1.5 kN/m^2 \times 2.3m = 3.5 kN/m$

Use 356 x 171 x 45 UB.



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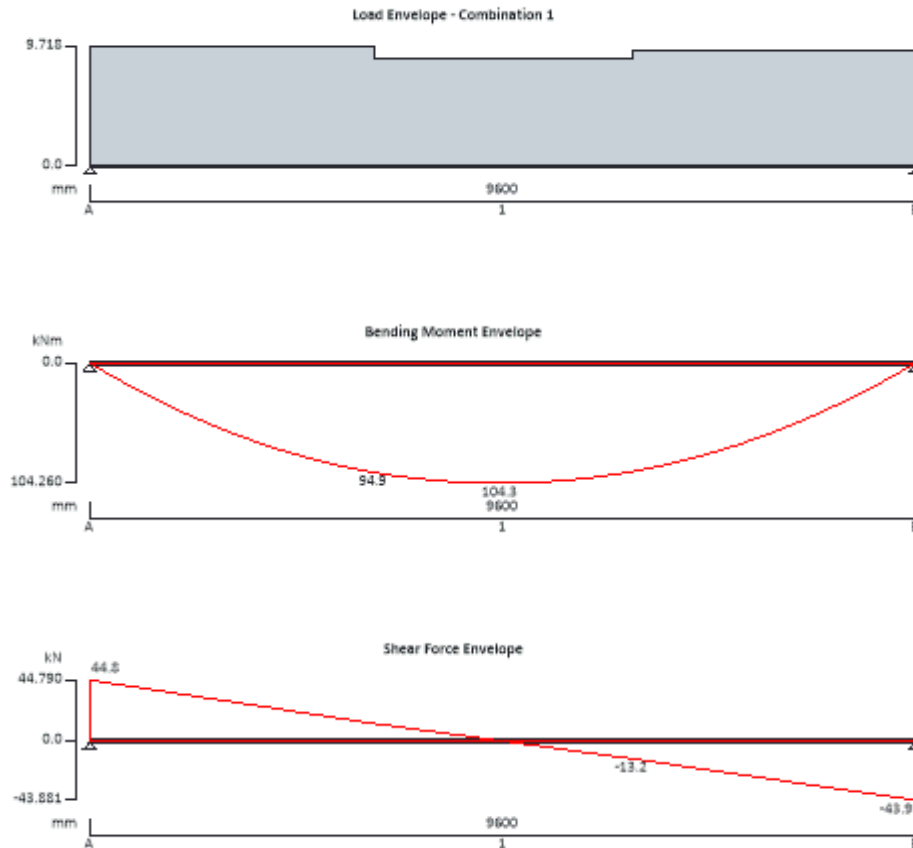
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STEEL BEAM ANALYSIS & DESIGN (BS5950)

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 $\times 1$
 Floor - Dead full UDL 1.2 kN/m
 Floor - Imposed full UDL 3.5 kN/m
 Stud Wall - Dead partial UDL 1.3 kN/m from 0 mm to 3300 mm
 Stud Wall - Dead partial UDL 0.53 kN/m from 3300 mm to 6300 mm
 Balustrade - Dead partial UDL 1 kN/m from 6300 mm to 9600 mm
 Dead full UDL 0 kN/m

Load combinations

Load combination 1

Support A

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



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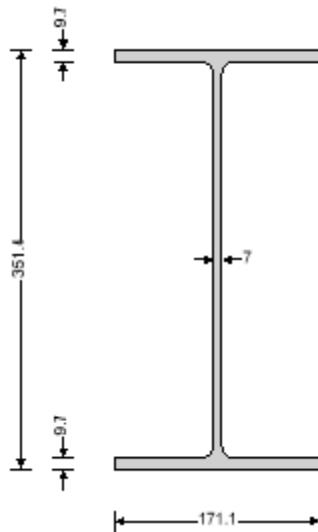
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Calcs by KA	Calcs date 17/11/2023	Checked by NF	Checked date 17/11/2023	Approved by NF	Approved date 17/11/2023

Analysis results

Maximum moment	$M_{max} = 104.3 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 44.8 \text{ kN}$	$V_{min} = -43.9 \text{ kN}$
Deflection	$\delta_{max} = 26.8 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 44.8 \text{ kN}$	$R_{A_min} = 44.8 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 12.8 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 16.8 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 43.9 \text{ kN}$	$R_{B_min} = 43.9 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 12.1 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 16.8 \text{ kN}$	

Section details

Section type	UKB 356x171x45 (Tata Steel Advance)
Steel grade	S355
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 9.7 \text{ mm}$
Design strength	$p_y = 355 \text{ N/mm}^2$
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

$$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = 0.88$$



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Internal compression parts - Table 11

Depth of section

$$d = 311.6 \text{ mm}$$

$$d / t = 50.6 \times \epsilon \leq 80 \times \epsilon$$

Class 1 plastic

Outstand flanges - Table 11

Width of section

$$b = B / 2 = 85.6 \text{ mm}$$

$$b / T = 10.0 \times \epsilon \leq 15 \times \epsilon$$

Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 44.8 \text{ kN}$$

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

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 2460 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 523.9 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 104.3 \text{ kNm}$$

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange

$$\beta_{2f} = 10 \times \epsilon = 8.801$$

Limiting value for class 3 semi-compact flange

$$\beta_{3f} = 15 \times \epsilon = 13.202$$

Limiting value for class 2 compact web

$$\beta_{2w} = 100 \times \epsilon = 88.014$$

Limiting value for class 3 semi-compact web

$$\beta_{3w} = 120 \times \epsilon = 105.617$$

Effective plastic modulus - cl.3.5.6.2

$$S_{eff} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \times \min(\frac{((\beta_{3w} / (d / t))^2 - 1)}{((\beta_{3w} / \beta_{2w})^2 - 1)}, \frac{((\beta_{3f} / (b / T) - 1)}{((\beta_{3f} / \beta_{2f} - 1))}), S_{xx}) = 774080 \text{ mm}^3$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z_{xx}) = 274.8 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 250 = 38.4 \text{ mm}$$

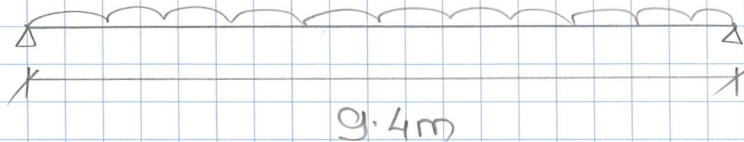
Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 26.806 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

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RIDGE STEEL BEAM - REAR BUILDING.



Design Span = 9.4m

Supporting;

$$\text{Roof} = \frac{5\text{m}}{2} = 2.5\text{m wide}$$

Loading;

$$G_k \rightarrow \text{Roof} = 1.1 \text{ kN/m}^2 \times 2.5\text{m} = 2.75 \text{ kN/m}$$

$$Q_k \rightarrow \text{Roof} = 0.6 \text{ kN/m}^2 \times 2.5\text{m} = 1.5 \text{ kN/m}$$

Use 305 x 165 x 40 UB.

$$R_{a, \text{dead}} = 14.8 \text{ kN}$$

$$R_{a, \text{live}} = 7 \text{ kN}$$

$$R_{a, \text{max}} = 32 \text{ kN}$$



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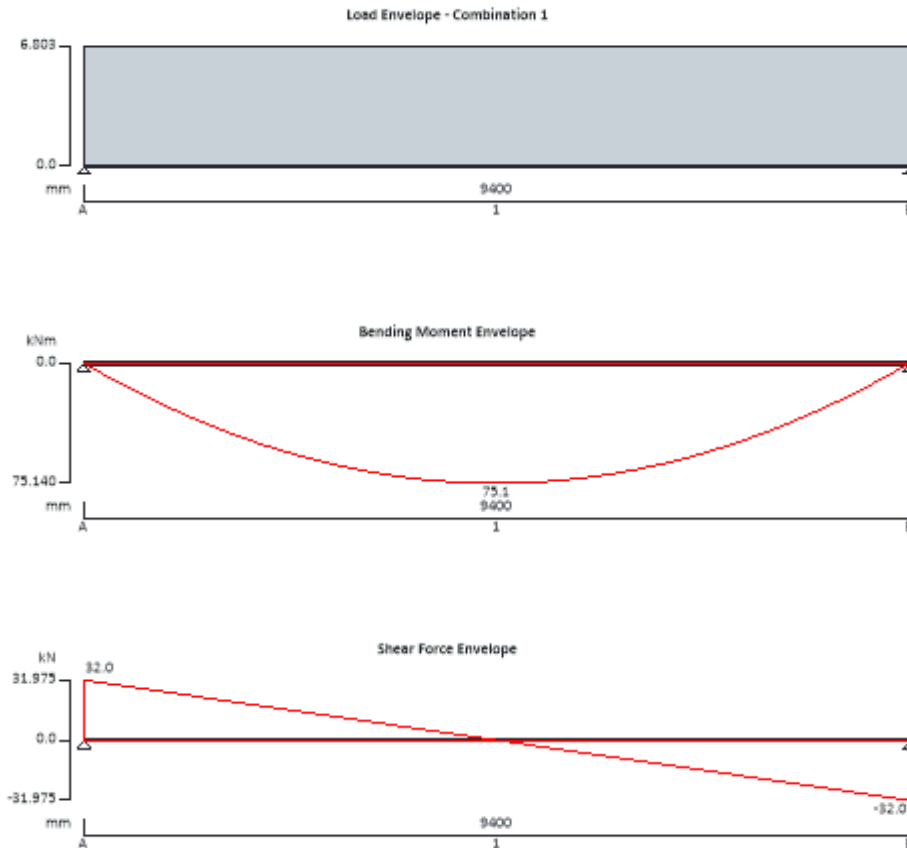
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Calcs for Ridge Steel Beam - Rear Building				Start page no./Revision 13	
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STEEL BEAM ANALYSIS & DESIGN (BS5950)

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 $\times 1$ Roof Rafter - Dead full UDL 2.75 kN/m Roof Rafter - Imposed full UDL 1.5 kN/m Dead full UDL 0 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 $\times 1.60$



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Analysis results

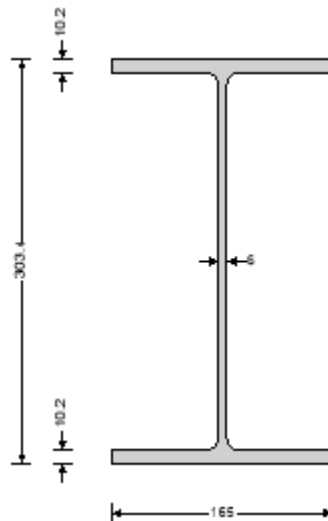
Maximum moment	$M_{max} = 75.1 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 32 \text{ kN}$	$V_{min} = -32 \text{ kN}$
Deflection	$\delta_{max} = 27.1 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 32 \text{ kN}$	$R_{A_min} = 32 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_Dead} = 14.8 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_Imposed} = 7.1 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 32 \text{ kN}$	$R_{B_min} = 32 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_Dead} = 14.8 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_Imposed} = 7 \text{ kN}$	

Section details

Section type	UKB 305x165x40 (Tata Steel Advance)
Steel grade	S275

From table 9: Design strength p_y

Thickness of element	$\max(T, t) = 10.2 \text{ mm}$
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	$d = 265.2 \text{ mm}$	
	$d / t = 44.2 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

Outstand flanges - Table 11

Width of section	$b = B / 2 = 82.5 \text{ mm}$
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$$b / T = 8.1 \times \varepsilon \leq 9 \times \varepsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{32 \text{ kN}}$$

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

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = \mathbf{1820 \text{ mm}^2}$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = \mathbf{300.4 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{75.1 \text{ kNm}}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{171.3 \text{ kNm}}$$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 250 = \mathbf{37.6 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{27.092 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit



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Calcs for

Floor Joists

Start page no./Revision

16

Calcs by

KA

Calcs date

17/11/2023

Checked by

NF

Checked date

17/11/2023

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NF

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17/11/2023

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

Joist details

Joist breadth

b = 50 mm

Joist depth

h = 200 mm

Joist spacing

s = 300 mm

Timber strength class

C24

Service class of timber

1



Span details

Number of spans

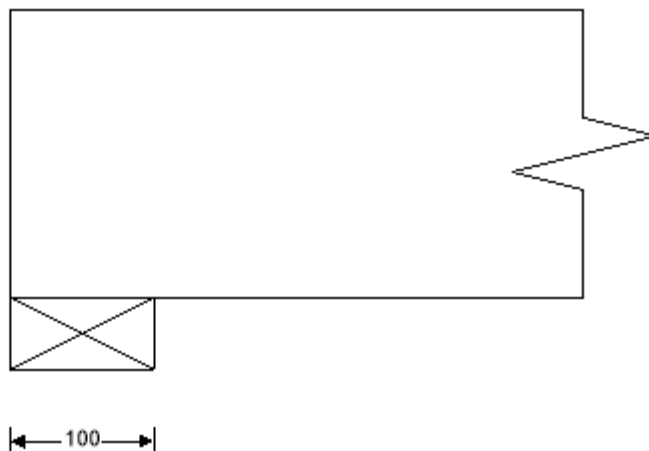
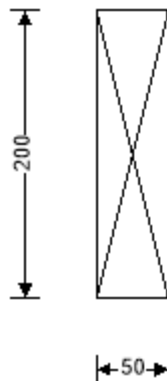
N_{span} = 1

Length of bearing

L_b = 100 mm

Effective length of span

L_{s1} = 4700 mm



Section properties

Second moment of area

I = b × h³ / 12 = 33333333 mm⁴

Section modulus

Z = b × h² / 6 = 333333 mm³

Loading details

Joist self weight

F_{swt} = b × h × ρ_{char} × g_{acc} = 0.03 kN/m

Dead load

F_{d_udl} = 0.50 kN/m²

Imposed UDL(Long term)

F_{i_udl} = 1.50 kN/m²

Imposed point load (Medium term)

F_{i_pt} = 1.40 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



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Service class for modulus of elasticity

$$K_{2e} = 1.00$$

Section depth factor

$$K_7 = 1.05$$

Load sharing factor

$$K_8 = 1.10$$

Consider long term loads

Load duration factor

$$K_3 = 1.00$$

Maximum bending moment

$$M = 1.752 \text{ kNm}$$

Maximum shear force

$$V = 1.491 \text{ kN}$$

Maximum support reaction

$$R = 1.491 \text{ kN}$$

Maximum deflection

$$\delta = 11.507 \text{ mm}$$

Check bending stress

Bending stress

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

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 8.626 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = M / Z = 5.255 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress

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

Permissible shear stress

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

Applied shear stress

$$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.224 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check bearing stress

Compression perpendicular to grain (no wane)

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

Permissible bearing stress

$$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = 2.640 \text{ N/mm}^2$$

Applied bearing stress

$$\sigma_{c_max} = R / (b \times L_b) = 0.298 \text{ N/mm}^2$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 14.000 \text{ mm}$$

Bending deflection (based on E_{mean})

$$\delta_{bending} = 11.195 \text{ mm}$$

Shear deflection

$$\delta_{shear} = 0.311 \text{ mm}$$

Total deflection

$$\delta = \delta_{bending} + \delta_{shear} = 11.507 \text{ mm}$$

PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor

$$K_3 = 1.25$$

Maximum bending moment

$$M = 2.154 \text{ kNm}$$

Maximum shear force

$$V = 1.833 \text{ kN}$$

Maximum support reaction

$$R = 1.833 \text{ kN}$$

Maximum deflection

$$\delta = 12.048 \text{ mm}$$

Check bending stress

Bending stress

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

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 10.783 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = M / Z = 6.462 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress

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

Permissible shear stress

$$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 0.976 \text{ N/mm}^2$$



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Applied shear stress

$$\tau_{\max} = 3 \times V / (2 \times b \times h) = \mathbf{0.275 \text{ N/mm}^2}$$

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

$$\sigma_{c_max} = R / (b \times L_b) = \mathbf{0.367 \text{ N/mm}^2}$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = \mathbf{14.000 \text{ mm}}$$

Bending deflection (based on E_{mean})

$$\delta_{bending} = \mathbf{11.665 \text{ mm}}$$

Shear deflection

$$\delta_{shear} = \mathbf{0.383 \text{ mm}}$$

Total deflection

$$\delta = \delta_{bending} + \delta_{shear} = \mathbf{12.048 \text{ mm}}$$

PASS - Actual deflection within permissible limits



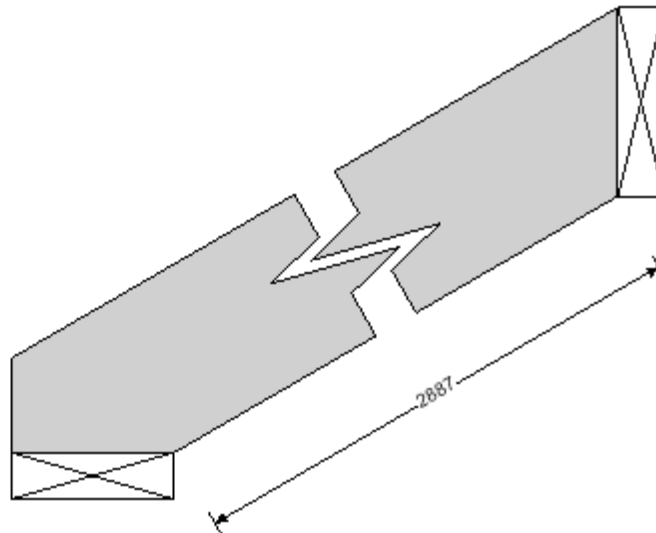
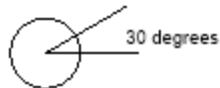
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TIMBER RAFTER DESIGN (BS5268-2:2002)

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

$b = 50 \text{ mm}$
 $h = 175 \text{ mm}$
 $s = 400 \text{ mm}$
 $\alpha = 30.0 \text{ deg}$
 $L_{clh} = 2500 \text{ mm}$
 $L_{cl} = L_{clh} / \cos(\alpha) = 2887 \text{ mm}$
Single span
C24

Section properties

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

$A = b \times h = 8750 \text{ mm}^2$
 $Z = b \times h^2 / 6 = 255208 \text{ mm}^3$
 $I = b \times h^3 / 12 = 22330729 \text{ mm}^4$
 $r = \sqrt{I / A} = 50.5 \text{ mm}$

Loading details

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

$F_j = b \times h \times \rho_{char} \times g_{acc} = 0.03 \text{ kN/m}$
 $F_d = 0.95 \text{ kN/m}^2$
 $F_u = 0.60 \text{ kN/m}^2$
 $F_p = 0.90 \text{ kN}$

Modification factors

Section depth factor
Load sharing factor

$K_7 = (300 \text{ mm} / h)^{0.11} = 1.06$
 $K_8 = 1.10$

Consider long term load condition

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

$K_3 = 1.00$
 $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = 0.355 \text{ kN/m}$
 $L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 4 \text{ mm}$
 $L_{eff} = L_{cl} + L_b = 2891 \text{ mm}$



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Check bending stress

Bending stress parallel to grain

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

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 8.754 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 1.453 \text{ N/mm}^2$$

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

$$K_{12} = 0.69$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 5.962 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.203 \text{ N/mm}^2$$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 21.704 \text{ N/mm}^2$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.990$$

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.202 < 1$$

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

$$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = 0.088 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = 8.672 \text{ mm}$$

Bending deflection

$$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = 1.339 \text{ mm}$$

Shear deflection

$$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = 0.075 \text{ mm}$$

Total deflection

$$\delta_{max} = \delta_b + \delta_s = 1.414 \text{ mm}$$

PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor

$$K_3 = 1.25$$

Total UDL perpendicular to rafter

$$F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) = 0.535 \text{ kN/m}$$

Notional bearing length

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

Effective span

$$L_{eff} = L_{cl} + L_b = 2893 \text{ mm}$$

Check bending stress

Bending stress parallel to grain

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

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 10.942 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 2.193 \text{ N/mm}^2$$

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

$$K_{12} = 0.66$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 7.146 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.306 \text{ N/mm}^2$$



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PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{21.674 \text{ N/mm}^2}$

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 $\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = \mathbf{0.246 < 1}$

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 $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = \mathbf{0.133 \text{ N/mm}^2}$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection $\delta_{adm} = 0.003 \times L_{eff} = \mathbf{8.678 \text{ mm}}$

Bending deflection $\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = \mathbf{2.023 \text{ mm}}$

Shear deflection $\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = \mathbf{0.114 \text{ mm}}$

Total deflection $\delta_{max} = \delta_b + \delta_s = \mathbf{2.136 \text{ mm}}$

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor $K_3 = \mathbf{1.50}$

Total UDL perpendicular to rafter $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = \mathbf{0.355 \text{ kN/m}}$

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 $L_{eff} = L_{cl} + L_b = \mathbf{2894 \text{ mm}}$

Check bending stress

Bending stress parallel to grain $\sigma_m = \mathbf{7.500 \text{ N/mm}^2}$

Permissible bending stress $\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{13.131 \text{ N/mm}^2}$

Applied bending stress $\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) + F_p \times \cos(\alpha) \times L_{eff} / (4 \times Z) = \mathbf{3.666 \text{ N/mm}^2}$

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 $K_{12} = \mathbf{0.63}$

Permissible compressive stress $\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{8.191 \text{ N/mm}^2}$

Applied compressive stress $\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) + F_p \times \sin(\alpha) / A = \mathbf{0.255 \text{ N/mm}^2}$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress $\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{21.660 \text{ N/mm}^2}$

Euler coefficient $K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = \mathbf{0.989}$

Combined axial compression and bending check $\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = \mathbf{0.313 < 1}$

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 $\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A) = \mathbf{0.222 \text{ N/mm}^2}$

PASS - Applied shear stress within permissible limits



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Check deflection

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = \mathbf{8.681 \text{ mm}}$$

Bending deflection

$$\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha) / 48) / (E_{mean} \times I) = \mathbf{2.975 \text{ mm}}$$

Shear deflection

$$\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{mean} \times A) = \mathbf{0.190 \text{ mm}}$$

Total deflection

$$\delta_{max} = \delta_b + \delta_s = \mathbf{3.165 \text{ mm}}$$

PASS - Total deflection within permissible limits



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MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.08

Masonry details

Masonry type

Compressive strength of unit

Mortar designation

Least horizontal dimension of masonry units

Height of masonry units

Category of masonry units

Category of construction control

Partial safety factor for material strength

Thickness of load bearing leaf

Effective thickness of masonry wall

Height of masonry wall

Effective height of masonry wall

Aggregate concrete blocks (25% or less formed voids)

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

iii

$l_{unit} = 100 \text{ mm}$

$h_{unit} = 215 \text{ mm}$

Category II

Normal

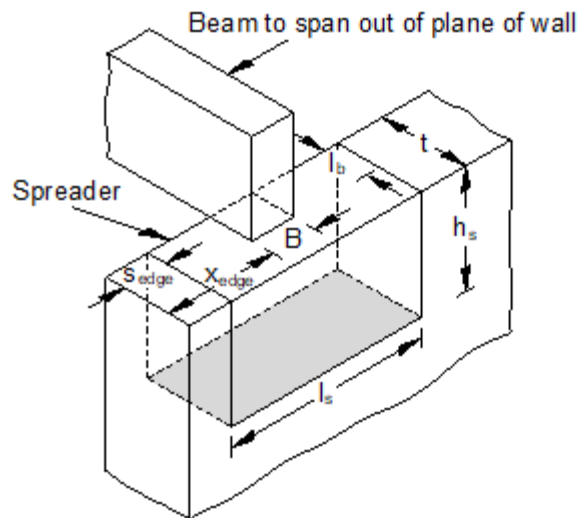
$\gamma_m = 3.5$

$t = 100 \text{ mm}$

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

$h = 2400 \text{ mm}$

$h_{ef} = 2400 \text{ mm}$



Bearing details

Beam spanning out of plane of wall

Width of bearing

$B = 100 \text{ mm}$

Length of bearing

$l_b = 100 \text{ mm}$

Edge distance

$X_{edge} = 100 \text{ mm}$

Compressive strength from Table 2 BS5628:Part 1 - aggregate concrete blocks (25% or less formed voids)

Mortar designation

Mortar = "iii"

Block compressive strength

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

Characteristic compressive strength (Table 2c)

$f_{kc} = 3.20 \text{ N/mm}^2$

Characteristic compressive strength (Table 2d)

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

Height of solid block

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

Least horizontal dimension

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



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Block ratio

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

Ratio between 0.6 and 4.5 - OK

Characteristic compressive strength

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

Loading details

Characteristic concentrated dead load

$$G_k = 14 \text{ kN}$$

Characteristic concentrated imposed load

$$Q_k = 34 \text{ kN}$$

Design concentrated load

$$F = (G_k \times 1.4) + (Q_k \times 1.6) = 74.4 \text{ kN}$$

Characteristic distributed dead load

$$g_k = 0.0 \text{ kN/m}$$

Characteristic distributed imposed load

$$q_k = 0.0 \text{ kN/m}$$

Design distributed load

$$f = (g_k \times 1.4) + (q_k \times 1.6) = 0.0 \text{ kN/m}$$

Masonry bearing type

Bearing type

Type 2

Bearing safety factor

$$\gamma_{\text{bear}} = 1.50$$

Check design bearing without a spreader

Design bearing stress

$$f_{ca} = F / (B \times l_b) + f / t = 7.442 \text{ N/mm}^2$$

Allowable bearing stress

$$f_{cp} = \gamma_{\text{bear}} \times f_k / \gamma_m = 2.743 \text{ N/mm}^2$$

FAIL - Design bearing stress exceeds allowable bearing stress, use a spreader

Spreader details

Length of spreader

$$l_s = 440 \text{ mm}$$

Depth of spreader

$$h_s = 215 \text{ mm}$$

Edge distance

$$x_{\text{edge}} = \max(0 \text{ mm}, (l_s - B) / 2) = 0 \text{ mm}$$

Spreader bearing type

Bearing type

Type 3

Bearing safety factor

$$\gamma_{\text{bear}} = 2.00$$

Check design bearing with a spreader

Loading acts eccentrically within middle third – triangular stress distribution

Eccentricity of load

$$e = ((l_s - B) / 2) - x_{\text{edge}} = 70 \text{ mm}$$

Maximum bearing stress

$$f_{ca} = F \times (1 + (6 \times e / l_s)) / (l_s \times t) + f / t = 3.306 \text{ N/mm}^2$$

Allowable bearing stress

$$f_{cp} = \gamma_{\text{bear}} \times f_k / \gamma_m = 3.657 \text{ N/mm}^2$$

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at $0.4 \times h$ below the bearing level

Slenderness ratio

$$h_{\text{ef}} / t_{\text{ef}} = 24.00$$

Eccentricity at top of wall

$$e_x = 16.7 \text{ mm}$$

From BS5628:1 Table 7

Capacity reduction factor

$$\beta = 0.39$$

Length of bearing distributed at $0.4 \times h$

$$l_d = 1160 \text{ mm}$$

Maximum bearing stress

$$f_{ca} = F / (l_d \times t) + f / t = 0.642 \text{ N/mm}^2$$

Allowable bearing stress

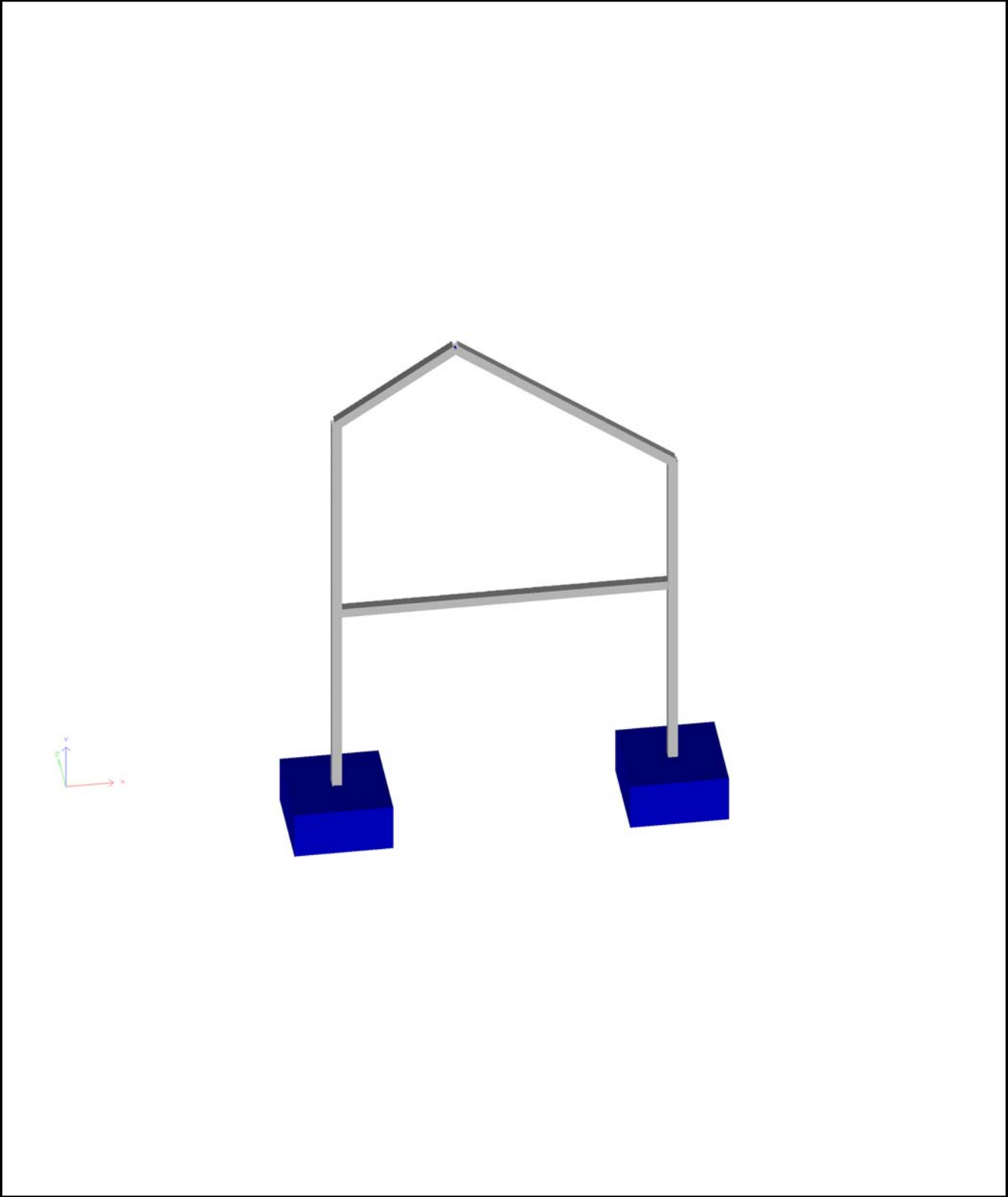
$$f_{cp} = \beta \times f_k / \gamma_m = 0.704 \text{ N/mm}^2$$

PASS - Allowable bearing stress at $0.4 \times h$ below bearing level exceeds design bearing stress

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		Designed By	KA
		Checked By	NF
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Current View



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Design Code

Design code	Concrete code	Steel Code	Wind Code
BS	BS8110-Amd.3	BS5950:2000	BS 6399-2

Members

Mbr ref	Member type	Start joint	Start fixity	End joint	End fixity	Orient (°)	Directional behaviour	Length (m)	P-Delta behaviour	Slope (°)
1	100x100x6.3 SHS	1	Pinned	6	Fixed	0.0	Normal	2.100	Normal	90.0
2	100x100x6.3 SHS	5	Pinned	7	Fixed	0.0	Normal	2.100	Normal	90.0
3	100x100x6.3 SHS	4	Fixed	3	Fixed	0.0	Normal	2.720	Normal	36.0
4	100x100x6.3 SHS	2	Fixed	3	Fixed	0.0	Normal	1.442	Normal	33.7
5	100x100x6.3 SHS	6	Fixed	2	Fixed	0.0	Normal	2.300	Normal	90.0
6	100x100x6.3 SHS	7	Fixed	4	Fixed	0.0	Normal	1.500	Normal	90.0
7	100x100x6.3 SHS	6	Fixed	8	Fixed	0.0	Normal	1.200	Normal	0.0
8	100x100x6.3 SHS	8	Fixed	7	Fixed	0.0	Normal	2.200	Normal	0.0

Joints

Joint ref	X pos (m)	Y pos (m)	Z pos (m)	Joint ref	X pos (m)	Y pos (m)	Z pos (m)	Joint ref	X pos (m)	Y pos (m)	Z pos (m)
1	0.000	0.000	0.000	2	0.000	4.400	0.000	3	1.200	5.200	0.000
4	3.400	3.600	0.000	5	3.400	0.000	0.000	6	0.000	2.100	0.000
7	3.400	2.100	0.000	8	1.200	2.100	0.000				

Supports

Joint ref	Support type	X Trans. (kN/mm)	Y Trans. (kN/mm)	Z Trans. (kN/mm)	X Rot. (kNm/Rad)	Y Rot. (kNm/Rad)	Z Rot. (kNm/Rad)	Direction control
1	Pinned Base	Fixed	Fixed	Fixed	Free	Fixed	Free	Normal
5	Pinned Base	Fixed	Fixed	Fixed	Free	Fixed	Free	Normal

Sections

Reference	Area (cm ²)	Ixx (cm ⁴)	Iyy (cm ⁴)	J (cm ⁴)	Elements (mm)				
					No	Width	Height	Vert. off	Lat. off
305x165 UKB40	51.3	8503	764	14.7					
100x100x5.0 SHS	18.7	279	279	439					
100x100x6.3 SHS	23.2	336	336	534					

Materials

Material reference	Elastic modulus (kN/mm ²)	Poisson ratio	Density (kN/m ³)	Thermal expansion (°Cx10-6)
UK-S355	205.00	0.30	77.00	12.00

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Member Types

Reference	Shape	Material	Seg no	Start section	End section	Length (m)	Depth (mm)	Placing rule
100x100x6.3 SHS	SW Library	UK-S355	1	100x100x6.3 SHS				

Self Weights

Mem ref	Self weight (kN)	Mem ref	Self weight (kN)	Mem ref	Self weight (kN)	Mem ref	Self weight (kN)
1	0.375	2	0.375	3	0.486	4	0.258
5	0.411	6	0.268	7	0.214	8	0.393
Total self weight for selected members : 2.780 kN							

Member Loads

Load reference	Load type	Start pos'n (m)	Start intensity (kN) & (m)	End pos'n (m)	End intensity (kN) & (m)	Direction	Category
Loads on member 7 (Length 1.200m)							
Glass Partial	UL		1.350			Vertical (-Y)	Dead
Loads on member 8 (Length 2.200m)							
Glass Partial	UL		1.150			Vertical (-Y)	Dead

Joint Loads

Load reference	Load type	Intensity (kN) & (m)	Direction	Category (type)
Loads on Joint 3				
Ridge Dead	JL	15.000	Vertical (-Y)	Dead (Dead)
Ridge Live	JL	7.000	Vertical (-Y)	Imposed (Imposed)

Load Combinations

Load Category			Partial Safety Factors					
No	Name	Type	Sub type	1	2	3	4	5
	Combination reference			Dead+Live	Dead+Live	Dead	Live	Comb5
	Limit state			ULS	SLS	SLS	SLS	ULS
	Elastic analysis			Linear	Linear	Linear	Linear	Linear
	Plastic analysis			No	No	No	No	No
1	Dead	Permanent	Permanent	1.40	1.00	1.00	0.00	0.00
2	Imposed	Variable	Cat B: office	1.60	1.00	0.00	1.00	0.00
3	Self Weight	Permanent	Self weight	1.40	1.00	1.00	0.00	0.00
4	Self Weight	Permanent	Self weight	0.00	0.00	0.00	0.00	0.00

Joint Displacements for Combination Dead+Live (ULS)

Analysis Type : Linear elastic

Joint reference	Displacements (mm)			Rotations (°)		
	Dx	Dy	Dz	Rx	Ry	Rz
1	0.00	0.00	0.00	0.000	0.000	-0.015
2	-10.99	-0.20	0.00	0.000	0.000	-0.205

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Joint Displacements for Combination Dead+Live (ULS) (continued)

Analysis Type : Linear elastic

Joint reference	Displacements (mm)			Rotations (°)		
	Dx	Dy	Dz	Rx	Ry	Rz
3	-5.34	-8.76	0.00	0.000	0.000	-0.231
4	0.87	-0.10	0.00	0.000	0.000	0.091
5	0.00	0.00	0.00	0.000	0.000	0.110
6	-1.76	-0.11	0.00	0.000	0.000	0.174
7	-1.71	-0.06	0.00	0.000	0.000	-0.079
8	-1.74	1.10	0.00	0.000	0.000	-0.009

Joint Displacements for Combination Dead+Live (SLS)

Analysis Type : Linear elastic

Joint reference	Displacements (mm)			Rotations (°)		
	Dx	Dy	Dz	Rx	Ry	Rz
1	0.00	0.00	0.00	0.000	0.000	-0.009
2	-7.51	-0.14	0.00	0.000	0.000	-0.140
3	-3.66	-5.98	0.00	0.000	0.000	-0.158
4	0.58	-0.07	0.00	0.000	0.000	0.062
5	0.00	0.00	0.00	0.000	0.000	0.074
6	-1.20	-0.07	0.00	0.000	0.000	0.118
7	-1.17	-0.04	0.00	0.000	0.000	-0.053
8	-1.19	0.71	0.00	0.000	0.000	-0.008

Joint Displacements for Combination Dead

Analysis Type : Linear elastic

Joint reference	Displacements (mm)			Rotations (°)		
	Dx	Dy	Dz	Rx	Ry	Rz
1	0.00	0.00	0.00	0.000	0.000	-0.002
2	-5.10	-0.10	0.00	0.000	0.000	-0.091
3	-2.52	-4.01	0.00	0.000	0.000	-0.107
4	0.32	-0.05	0.00	0.000	0.000	0.041
5	0.00	0.00	0.00	0.000	0.000	0.046
6	-0.82	-0.05	0.00	0.000	0.000	0.071
7	-0.79	-0.03	0.00	0.000	0.000	-0.027
8	-0.81	0.19	0.00	0.000	0.000	-0.015

Joint Displacements for Combination Live

Analysis Type : Linear elastic

Joint reference	Displacements (mm)			Rotations (°)		
	Dx	Dy	Dz	Rx	Ry	Rz
1	0.00	0.00	0.00	0.000	0.000	-0.008
2	-2.41	-0.04	0.00	0.000	0.000	-0.049
3	-1.13	-1.97	0.00	0.000	0.000	-0.051
4	0.26	-0.02	0.00	0.000	0.000	0.021

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Joint Displacements for Combination Live (continued)

Analysis Type : Linear elastic

Joint reference	Displacements (mm)			Rotations (°)		
	Dx	Dy	Dz	Rx	Ry	Rz
5	0.00	0.00	0.00	0.000	0.000	0.028
6	-0.39	-0.02	0.00	0.000	0.000	0.047
7	-0.38	-0.01	0.00	0.000	0.000	-0.026
8	-0.38	0.52	0.00	0.000	0.000	0.007

Support Reactions for Combination Dead+Live (ULS)

Analysis Type : Linear elastic

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	-1.029	23.849	0.000	0.000	0.000	0.000
5	1.029	14.161	0.000	0.000	0.000	0.000

Support Reactions for Combination Dead+Live (SLS)

Analysis Type : Linear elastic

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	-0.693	16.388	0.000	0.000	0.000	0.000
5	0.693	9.762	0.000	0.000	0.000	0.000

Support Reactions for Combination Dead

Analysis Type : Linear elastic

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	-0.394	11.859	0.000	0.000	0.000	0.000
5	0.394	7.291	0.000	0.000	0.000	0.000

Support Reactions for Combination Live

Analysis Type : Linear elastic

Joint reference	Support reactions (kN)			Support moments (kNm)		
	Px	Py	Pz	Mx	My	Mz
1	-0.298	4.529	0.000	0.000	0.000	0.000
5	0.298	2.471	0.000	0.000	0.000	0.000

Maximum Axial, Shear and Deflection effects for Combination Dead+Live (ULS)

Member ref	Axial effects		Shear effects		Deflection effects					
	Compr'n (kN)	Tension (kN)	Normal (kN)	Lateral (kN)	Axial (mm)	Pos (m)	Normal (mm)	Pos (m)	Lateral (mm)	Pos (m)
1	23.849	0.000	1.029	0.000	-0.11	2.100	1.76	2.100	0.00	0.000
2	14.161	0.000	-1.029	0.000	-0.06	2.100	2.04	1.600	0.00	0.000
3	12.455	0.000	5.745	0.000	-0.83	2.720	-10.89	2.384	0.00	0.000
4	16.790	0.000	13.118	0.000	-9.31	1.442	5.93	0.000	0.00	0.000

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Maximum Axial, Shear and Deflection effects for Combination Dead+Live (ULS) (continued)

Member ref	Axial effects		Shear effects		Deflection effects					
	Compr'n (kN)	Tension (kN)	Normal (kN)	Lateral (kN)	Axial (mm)	Pos (m)	Normal (mm)	Pos (m)	Lateral (mm)	Pos (m)
5	20.229	0.000	-6.694	0.000	-0.20	2.300	11.48	2.018	0.00	0.000
6	11.971	0.000	6.694	0.000	-0.10	1.500	1.71	0.000	0.00	0.000
7	0.000	-7.723	3.620	0.000	-1.76	0.000	1.11	1.037	0.00	0.000
8	0.000	-7.723	-2.190	0.000	-1.74	0.000	1.10	0.000	0.00	0.000

Maximum Axial, Shear and Deflection effects for Combination Dead+Live (SLS)

Member ref	Axial effects		Shear effects		Deflection effects					
	Compr'n (kN)	Tension (kN)	Normal (kN)	Lateral (kN)	Axial (mm)	Pos (m)	Normal (mm)	Pos (m)	Lateral (mm)	Pos (m)
1	16.388	0.000	0.693	0.000	-0.07	2.100	1.20	2.100	0.00	0.000
2	9.762	0.000	-0.693	0.000	-0.04	2.100	1.38	1.605	0.00	0.000
3	8.517	0.000	3.920	0.000	-0.56	2.720	-7.43	2.384	0.00	0.000
4	11.479	0.000	8.957	0.000	-6.36	1.442	4.05	0.000	0.00	0.000
5	13.820	0.000	-4.582	0.000	-0.14	2.300	7.84	2.019	0.00	0.000
6	8.180	0.000	4.582	0.000	-0.07	1.500	1.17	0.000	0.00	0.000
7	0.000	-5.275	2.568	0.000	-1.20	0.000	0.73	1.001	0.00	0.000
8	0.000	-5.275	-1.582	0.000	-1.19	0.000	0.71	0.000	0.00	0.000

Maximum Axial, Shear and Deflection effects for Combination Dead

Member ref	Axial effects		Shear effects		Deflection effects					
	Compr'n (kN)	Tension (kN)	Normal (kN)	Lateral (kN)	Axial (mm)	Pos (m)	Normal (mm)	Pos (m)	Lateral (mm)	Pos (m)
1	11.859	0.000	0.394	0.000	-0.05	2.100	0.82	2.100	0.00	0.000
2	7.291	0.000	-0.394	0.000	-0.03	2.100	0.89	1.669	0.00	0.000
3	5.861	0.000	2.639	0.000	-0.32	2.720	-5.03	2.382	0.00	0.000
4	7.877	0.000	6.069	0.000	-4.32	1.442	2.75	0.000	0.00	0.000
5	9.419	0.000	-3.188	0.000	-0.10	2.300	5.31	2.033	0.00	0.000
6	5.581	0.000	3.188	0.000	-0.05	1.500	0.79	0.000	0.00	0.000
7	0.000	-3.582	2.440	0.000	-0.82	0.000	0.28	0.657	0.00	0.000
8	0.000	-3.582	-1.710	0.000	-0.81	0.000	0.19	0.000	0.00	0.000

Maximum Axial, Shear and Deflection effects for Combination Live

Member ref	Axial effects		Shear effects		Deflection effects					
	Compr'n (kN)	Tension (kN)	Normal (kN)	Lateral (kN)	Axial (mm)	Pos (m)	Normal (mm)	Pos (m)	Lateral (mm)	Pos (m)
1	4.529	0.000	0.298	0.000	-0.02	2.100	0.39	2.100	0.00	0.000
2	2.471	0.000	-0.298	0.000	-0.01	2.100	0.50	1.515	0.00	0.000
3	2.656	0.000	1.281	0.000	-0.24	2.720	-2.40	2.386	0.00	0.000
4	3.602	0.000	2.889	0.000	-2.04	1.442	1.30	0.000	0.00	0.000
5	4.402	0.000	-1.394	0.000	-0.04	2.300	2.53	1.989	0.00	0.000
6	2.598	0.000	1.394	0.000	-0.02	1.500	0.38	0.000	0.00	0.000

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Maximum Axial, Shear and Deflection effects for Combination Live (continued)

Member ref	Axial effects		Shear effects		Deflection effects					
	Compr'n (kN)	Tension (kN)	Normal (kN)	Lateral (kN)	Axial (mm)	Pos (m)	Normal (mm)	Pos (m)	Lateral (mm)	Pos (m)
7	0.000	-1.693	0.128	0.000	-0.39	0.000	0.52	1.200	0.00	0.000
8	0.000	-1.693	0.128	0.000	-0.38	0.000	0.54	0.281	0.00	0.000

Member Deflections (Global) for combination Dead+Live (ULS)

Analysis Type : Linear elastic

Interval no	Interval pos. (m)	Displacements (mm)			Slopes (°)		
		Axial	Normal	Lateral	Torsion	Normal	Lateral
Member 1							
0	0.000	0.00	0.00	0.00	0.000	-0.015	0.000
1	0.525	-0.03	-0.10	0.00	0.000	-0.003	0.000
2	1.050	-0.05	0.02	0.00	0.000	0.032	0.000
3	1.575	-0.08	0.57	0.00	0.000	0.091	0.000
4	2.100	-0.11	1.76	0.00	0.000	0.174	0.000
Member 2							
0	0.000	0.00	0.00	0.00	0.000	0.110	0.000
1	0.525	-0.02	0.97	0.00	0.000	0.098	0.000
2	1.050	-0.03	1.72	0.00	0.000	0.062	0.000
3	1.575	-0.05	2.04	0.00	0.000	0.003	0.000
4	2.100	-0.06	1.71	0.00	0.000	-0.079	0.000
Member 3							
0	0.000	-0.76	0.43	0.00	0.000	-0.091	0.000
1	0.680	-0.78	-2.36	0.00	0.000	-0.342	0.000
2	1.360	-0.80	-6.82	0.00	0.000	-0.372	0.000
3	2.040	-0.81	-10.31	0.00	0.000	-0.181	0.000
4	2.720	-0.83	-10.23	0.00	0.000	0.231	0.000
Member 4							
0	0.000	-9.26	5.93	0.00	0.000	-0.205	0.000
1	0.361	-9.27	3.87	0.00	0.000	-0.424	0.000
2	0.721	-9.28	0.88	0.00	0.000	-0.502	0.000
3	1.082	-9.30	-2.15	0.00	0.000	-0.438	0.000
4	1.442	-9.31	-4.33	0.00	0.000	-0.231	0.000
Member 5							
0	0.000	-0.11	1.76	0.00	0.000	0.174	0.000
1	0.575	-0.13	4.57	0.00	0.000	0.355	0.000
2	1.150	-0.15	8.28	0.00	0.000	0.353	0.000
3	1.725	-0.18	11.03	0.00	0.000	0.166	0.000
4	2.300	-0.20	10.99	0.00	0.000	-0.205	0.000
Member 6							
0	0.000	-0.06	1.71	0.00	0.000	-0.079	0.000
1	0.375	-0.07	0.90	0.00	0.000	-0.154	0.000
2	0.750	-0.08	-0.14	0.00	0.000	-0.151	0.000
3	1.125	-0.09	-0.90	0.00	0.000	-0.069	0.000

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Member Deflections (Global) for combination Dead+Live (ULS) (continued)

Analysis Type : Linear elastic

Interval no	Interval pos. (m)	Displacements (mm)			Slopes (°)		
		Axial	Normal	Lateral	Torsion	Normal	Lateral
4	1.500	-0.10	-0.87	0.00	0.000	0.091	0.000
Member 7							
0	0.000	-1.76	-0.11	0.00	0.000	0.174	0.000
1	0.300	-1.76	0.60	0.00	0.000	0.098	0.000
2	0.600	-1.75	0.96	0.00	0.000	0.045	0.000
3	0.900	-1.75	1.10	0.00	0.000	0.011	0.000
4	1.200	-1.74	1.10	0.00	0.000	-0.009	0.000
Member 8							
0	0.000	-1.74	1.10	0.00	0.000	-0.009	0.000
1	0.550	-1.73	0.93	0.00	0.000	-0.022	0.000
2	1.100	-1.73	0.71	0.00	0.000	-0.023	0.000
3	1.650	-1.72	0.45	0.00	0.000	-0.035	0.000
4	2.200	-1.71	-0.06	0.00	0.000	-0.079	0.000

Member Deflections (Global) for combination Dead+Live (SLS)

Analysis Type : Linear elastic

Interval no	Interval pos. (m)	Displacements (mm)			Slopes (°)		
		Axial	Normal	Lateral	Torsion	Normal	Lateral
Member 1							
0	0.000	0.00	0.00	0.00	0.000	-0.009	0.000
1	0.525	-0.02	-0.06	0.00	0.000	-0.002	0.000
2	1.050	-0.04	0.02	0.00	0.000	0.022	0.000
3	1.575	-0.05	0.39	0.00	0.000	0.062	0.000
4	2.100	-0.07	1.20	0.00	0.000	0.118	0.000
Member 2							
0	0.000	0.00	0.00	0.00	0.000	0.074	0.000
1	0.525	-0.01	0.66	0.00	0.000	0.066	0.000
2	1.050	-0.02	1.17	0.00	0.000	0.042	0.000
3	1.575	-0.03	1.38	0.00	0.000	0.003	0.000
4	2.100	-0.04	1.17	0.00	0.000	-0.053	0.000
Member 3							
0	0.000	-0.51	0.29	0.00	0.000	-0.062	0.000
1	0.680	-0.52	-1.62	0.00	0.000	-0.233	0.000
2	1.360	-0.54	-4.66	0.00	0.000	-0.254	0.000
3	2.040	-0.55	-7.04	0.00	0.000	-0.123	0.000
4	2.720	-0.56	-6.99	0.00	0.000	0.158	0.000
Member 4							
0	0.000	-6.32	4.05	0.00	0.000	-0.140	0.000
1	0.361	-6.33	2.65	0.00	0.000	-0.289	0.000
2	0.721	-6.34	0.61	0.00	0.000	-0.343	0.000
3	1.082	-6.35	-1.46	0.00	0.000	-0.299	0.000
4	1.442	-6.36	-2.95	0.00	0.000	-0.158	0.000

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Member Deflections (Global) for combination Dead+Live (SLS) (continued)

Analysis Type : Linear elastic

Interval no	Interval pos. (m)	Displacements (mm)			Slopes (°)		
		Axial	Normal	Lateral	Torsion	Normal	Lateral
Member 5							
0	0.000	-0.07	1.20	0.00	0.000	0.118	0.000
1	0.575	-0.09	3.12	0.00	0.000	0.242	0.000
2	1.150	-0.11	5.65	0.00	0.000	0.241	0.000
3	1.725	-0.12	7.53	0.00	0.000	0.114	0.000
4	2.300	-0.14	7.51	0.00	0.000	-0.140	0.000
Member 6							
0	0.000	-0.04	1.17	0.00	0.000	-0.053	0.000
1	0.375	-0.05	0.62	0.00	0.000	-0.105	0.000
2	0.750	-0.06	-0.08	0.00	0.000	-0.103	0.000
3	1.125	-0.06	-0.60	0.00	0.000	-0.047	0.000
4	1.500	-0.07	-0.58	0.00	0.000	0.062	0.000
Member 7							
0	0.000	-1.20	-0.07	0.00	0.000	0.118	0.000
1	0.300	-1.20	0.40	0.00	0.000	0.065	0.000
2	0.600	-1.20	0.64	0.00	0.000	0.029	0.000
3	0.900	-1.19	0.72	0.00	0.000	0.005	0.000
4	1.200	-1.19	0.71	0.00	0.000	-0.008	0.000
Member 8							
0	0.000	-1.19	0.71	0.00	0.000	-0.008	0.000
1	0.550	-1.18	0.59	0.00	0.000	-0.015	0.000
2	1.100	-1.18	0.45	0.00	0.000	-0.014	0.000
3	1.650	-1.17	0.29	0.00	0.000	-0.022	0.000
4	2.200	-1.17	-0.04	0.00	0.000	-0.053	0.000

Member Deflections (Global) for combination Dead

Analysis Type : Linear elastic

Interval no	Interval pos. (m)	Displacements (mm)			Slopes (°)		
		Axial	Normal	Lateral	Torsion	Normal	Lateral
Member 1							
0	0.000	0.00	0.00	0.00	0.000	-0.002	0.000
1	0.525	-0.01	0.00	0.00	0.000	0.003	0.000
2	1.050	-0.03	0.08	0.00	0.000	0.016	0.000
3	1.575	-0.04	0.32	0.00	0.000	0.039	0.000
4	2.100	-0.05	0.82	0.00	0.000	0.071	0.000
Member 2							
0	0.000	0.00	0.00	0.00	0.000	0.046	0.000
1	0.525	-0.01	0.40	0.00	0.000	0.041	0.000
2	1.050	-0.02	0.73	0.00	0.000	0.028	0.000
3	1.575	-0.02	0.88	0.00	0.000	0.005	0.000
4	2.100	-0.03	0.79	0.00	0.000	-0.027	0.000
Member 3							

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Member Deflections (Global) for combination Dead (continued)

Analysis Type : Linear elastic

Interval no	Interval pos. (m)	Displacements (mm)			Slopes (°)		
		Axial	Normal	Lateral	Torsion	Normal	Lateral
0	0.000	-0.29	0.15	0.00	0.000	-0.041	0.000
1	0.680	-0.30	-1.13	0.00	0.000	-0.157	0.000
2	1.360	-0.30	-3.17	0.00	0.000	-0.170	0.000
3	2.040	-0.31	-4.77	0.00	0.000	-0.083	0.000
4	2.720	-0.32	-4.73	0.00	0.000	0.107	0.000
Member 4							
0	0.000	-4.30	2.75	0.00	0.000	-0.091	0.000
1	0.361	-4.31	1.82	0.00	0.000	-0.193	0.000
2	0.721	-4.31	0.45	0.00	0.000	-0.230	0.000
3	1.082	-4.32	-0.94	0.00	0.000	-0.201	0.000
4	1.442	-4.32	-1.94	0.00	0.000	-0.107	0.000
Member 5							
0	0.000	-0.05	0.82	0.00	0.000	0.071	0.000
1	0.575	-0.06	2.06	0.00	0.000	0.162	0.000
2	1.150	-0.08	3.77	0.00	0.000	0.165	0.000
3	1.725	-0.09	5.08	0.00	0.000	0.081	0.000
4	2.300	-0.10	5.10	0.00	0.000	-0.091	0.000
Member 6							
0	0.000	-0.03	0.79	0.00	0.000	-0.027	0.000
1	0.375	-0.04	0.47	0.00	0.000	-0.066	0.000
2	0.750	-0.04	0.01	0.00	0.000	-0.067	0.000
3	1.125	-0.05	-0.33	0.00	0.000	-0.032	0.000
4	1.500	-0.05	-0.32	0.00	0.000	0.041	0.000
Member 7							
0	0.000	-0.82	-0.05	0.00	0.000	0.071	0.000
1	0.300	-0.81	0.20	0.00	0.000	0.029	0.000
2	0.600	-0.81	0.28	0.00	0.000	0.003	0.000
3	0.900	-0.81	0.26	0.00	0.000	-0.010	0.000
4	1.200	-0.81	0.19	0.00	0.000	-0.015	0.000
Member 8							
0	0.000	-0.81	0.19	0.00	0.000	-0.015	0.000
1	0.550	-0.80	0.07	0.00	0.000	-0.009	0.000
2	1.100	-0.80	0.03	0.00	0.000	0.002	0.000
3	1.650	-0.80	0.06	0.00	0.000	0.001	0.000
4	2.200	-0.79	-0.03	0.00	0.000	-0.027	0.000

Member Deflections (Global) for combination Live

Analysis Type : Linear elastic

Interval no	Interval pos. (m)	Displacements (mm)			Slopes (°)		
		Axial	Normal	Lateral	Torsion	Normal	Lateral
Member 1							
0	0.000	0.00	0.00	0.00	0.000	-0.008	0.000

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		Job Ref	Steel Frame
		Designed By	KA
		Checked By	NF
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Member Deflections (Global) for combination Live (continued)

Analysis Type : Linear elastic

Interval no	Interval pos. (m)	Displacements (mm)			Slopes (°)		
		Axial	Normal	Lateral	Torsion	Normal	Lateral
1	0.525	0.00	-0.06	0.00	0.000	-0.004	0.000
2	1.050	-0.01	-0.06	0.00	0.000	0.006	0.000
3	1.575	-0.01	0.07	0.00	0.000	0.023	0.000
4	2.100	-0.02	0.39	0.00	0.000	0.047	0.000
Member 2							
0	0.000	0.00	0.00	0.00	0.000	0.028	0.000
1	0.525	0.00	0.25	0.00	0.000	0.025	0.000
2	1.050	-0.01	0.44	0.00	0.000	0.015	0.000
3	1.575	-0.01	0.50	0.00	0.000	-0.002	0.000
4	2.100	-0.01	0.38	0.00	0.000	-0.026	0.000
Member 3							
0	0.000	-0.22	0.14	0.00	0.000	-0.021	0.000
1	0.680	-0.23	-0.49	0.00	0.000	-0.077	0.000
2	1.360	-0.23	-1.49	0.00	0.000	-0.083	0.000
3	2.040	-0.24	-2.27	0.00	0.000	-0.041	0.000
4	2.720	-0.24	-2.26	0.00	0.000	0.051	0.000
Member 4							
0	0.000	-2.02	1.30	0.00	0.000	-0.049	0.000
1	0.361	-2.03	0.83	0.00	0.000	-0.096	0.000
2	0.721	-2.03	0.15	0.00	0.000	-0.113	0.000
3	1.082	-2.03	-0.52	0.00	0.000	-0.098	0.000
4	1.442	-2.04	-1.01	0.00	0.000	-0.051	0.000
Member 5							
0	0.000	-0.02	0.39	0.00	0.000	0.047	0.000
1	0.575	-0.03	1.06	0.00	0.000	0.081	0.000
2	1.150	-0.03	1.88	0.00	0.000	0.076	0.000
3	1.725	-0.04	2.45	0.00	0.000	0.033	0.000
4	2.300	-0.04	2.41	0.00	0.000	-0.049	0.000
Member 6							
0	0.000	-0.01	0.38	0.00	0.000	-0.026	0.000
1	0.375	-0.01	0.15	0.00	0.000	-0.039	0.000
2	0.750	-0.02	-0.10	0.00	0.000	-0.035	0.000
3	1.125	-0.02	-0.27	0.00	0.000	-0.015	0.000
4	1.500	-0.02	-0.26	0.00	0.000	0.021	0.000
Member 7							
0	0.000	-0.39	-0.02	0.00	0.000	0.047	0.000
1	0.300	-0.39	0.20	0.00	0.000	0.036	0.000
2	0.600	-0.39	0.35	0.00	0.000	0.025	0.000
3	0.900	-0.38	0.46	0.00	0.000	0.016	0.000
4	1.200	-0.38	0.52	0.00	0.000	0.007	0.000
Member 8							
0	0.000	-0.38	0.52	0.00	0.000	0.007	0.000

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Member Deflections (Global) for combination Live (continued)

Analysis Type : Linear elastic

Interval no	Interval pos. (m)	Displacements (mm)			Slopes (°)		
		Axial	Normal	Lateral	Torsion	Normal	Lateral
1	0.550	-0.38	0.52	0.00	0.000	-0.006	0.000
2	1.100	-0.38	0.42	0.00	0.000	-0.016	0.000
3	1.650	-0.38	0.23	0.00	0.000	-0.023	0.000
4	2.200	-0.38	-0.01	0.00	0.000	-0.026	0.000

Steelwork Design Results

Design object ref	Memb ref	Design parameter	Analysis section	Design section	Utilisation factors				Design status
					Local capacity	Lateral buckling	Torsion	Deflection	
Group Reference - Steel Frame									
2	1-5	defaults	100x100x6.3 SHS		0.339	0.254	n / a	0.341	Passed
3	2-6	defaults	100x100x6.3 SHS		0.223	0.137	n / a	0.092	Passed
4	7-8	defaults	100x100x6.3 SHS		0.124	0.054	n / a	0.046	Passed
5	3	defaults	100x100x6.3 SHS		0.323	0.171	n / a	0.535	Passed
6	4	defaults	100x100x6.3 SHS		0.339	0.170	n / a	0.970	Passed

Design Results Summary

Group reference	Design object			Comments	Design status
	Reference	Type	Code		
Steel Frame	2	SW Design	BS5950:2000		Passed
	3	SW Design	BS5950:2000		Passed
	4	SW Design	BS5950:2000		Passed
	5	SW Design	BS5950:2000		Passed
	6	SW Design	BS5950:2000		Passed