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STRUCTURAL CALCULATIONS

Proposed Garage
Westwick Lock House, Boroughbridge

REF No. 1103_101

August 2022

Design Statement

General Notes

All calculations within this document are designed in accordance with all necessary regulations and codes of practice. All materials specified within are to be used strictly in accordance with manufacturer's recommendations and current codes of practice.

Codes Used

- NHBC
- BS 648: 1964 - Weights of Building Materials
- BS 6399: Pt 1: 1996 - Loadings for Buildings
- BS 6399: Pt 3: 1988 - Imposed Roof Loadings
- BS 5950: Pt 1: 2000 - Structural Steel
- BS 5628: Pt 1: 2005 - Masonry
- BS 5268: Pt 2: 2002 - Timber
- BS 8110: Pt 1: 1997 - Concrete

All works are to be carried out in accordance with all relevant CDM regulations, the health and safety guidelines and building regulations

Prior to any demolition of the existing structure an inspection is to be carried out to confirm any load bearing walls.

Nominal door and window openings to have standard steel or precast concrete lintels to suit.

All steelwork to be in accordance with CE marking standards, minimum CC2 and EXC2 classes

All steelwork to be minimum grade S355 unless specified.

All steelwork connections to be agreed with fabricator.

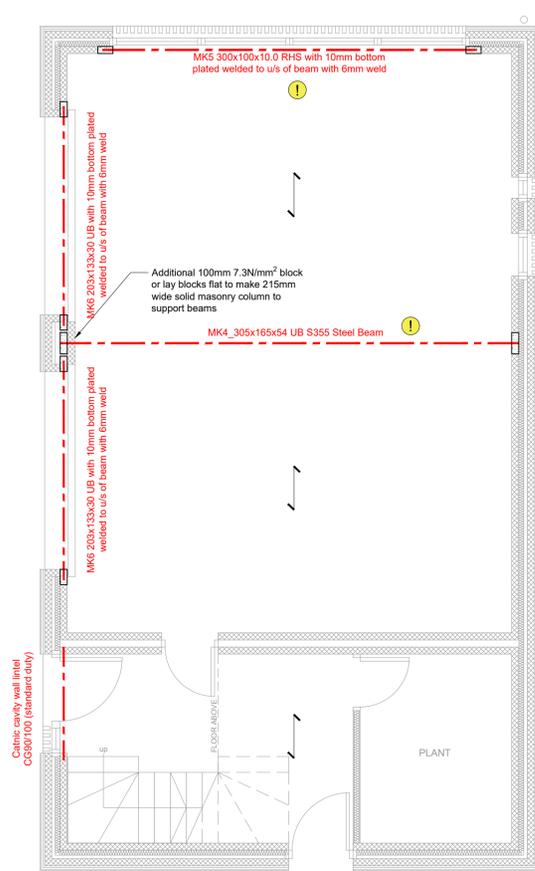
Steelwork to have minimum 2No. coats of high build zinc phosphate primer.

All floor timber to have solid timber noggins at maximum 1800mm centers

Report No: 1103_101– Structural Calculations
Project Details: Westwick Lock House, Boroughbridge
Date: August 2022



Design Summary

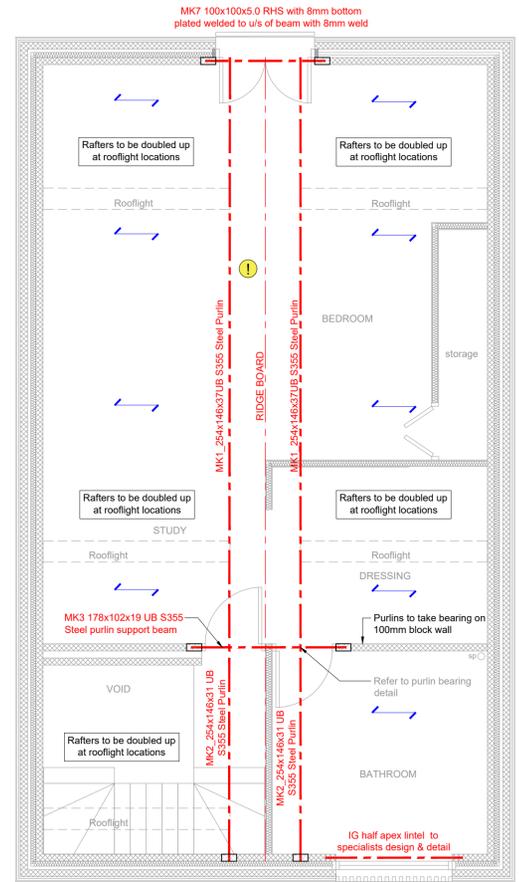


PROPOSED GROUND FLOOR SUPERSTRUCTURE PLAN

KEY

	Min. 215x100x140mm deep C30 concrete padstone
	Min. 300x100x140mm deep C30 concrete padstone
	Span of proposed 50x150 C16 timber rafters @ 400mm CTRS
	Span of proposed 50x200 C24 timber first floor joists @ 400mm CTRS
	Steel providers/contractors to complete risk assessment of large span/heavy beams before ordering
Beams to have minimum bearing of 100mm perpendicular to a wall and 150mm in the plane of a wall	

Existing foundations & ground floor walls as built. Foundation & masonry design by others.



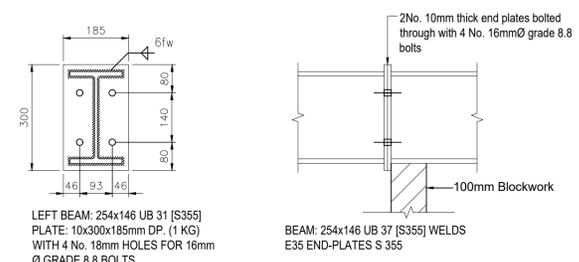
PROPOSED FIRST FLOOR SUPERSTRUCTURE PLAN

GENERAL NOTES

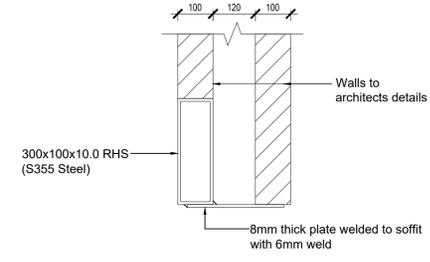
- Drawing may not be copied without written permission.
- Contractor to check all dimensions and report all errors and omissions to the engineer. Do NOT scale from this drawing.
- All works are to be carried out in accordance with all relevant CDM regulations, the health and safety guidelines and building regulations
- Prior to any demolition of the existing structure an inspection is to be carried out to confirm any load bearing walls.
- Nominal door and window openings to have standard steel or precast concrete lintels to suit.
- All steelwork to be in accordance with CE marking standards, minimum CC2 and EXC2 classes
- All steelwork to be minimum grade S355 unless specified.
- All steelwork connections to be agreed with fabricator.
- Steelwork to have minimum 2No. coats of high build zinc phosphate primer.
- All floor timber to have solid timber noggins at maximum 1800mm centers
- All beams to have minimum 215x100x140 deep c30 precast concrete padstones U.N.O

GENERAL NOTES

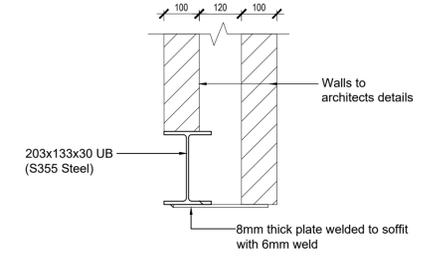
- ALL WORK TO BE CARRIED OUT IN STRICT ACCORDANCE WITH THE BUILDING REGULATIONS AND TO THE SATISFACTION OF THE LOCAL AUTHORITY TOWN/COUNTRY PLANNING, BUILDING CONTROL AND DRAINAGE DEPARTMENTS
- APPOINTED CONTRACTOR RESPONSIBLE FOR NOTIFYING LOCAL AUTHORITY BUILDING CONTROL DEPARTMENT UPON COMMENCEMENT OF BUILDING WORKS ON SITE.
- DIMENSIONS ALL TO SITE CHECK. DISCREPANCIES (IF ANY) TO BE BROUGHT TO THE IMMEDIATE ATTENTION OF THE DESIGNER.
- THESE PLANS HAVE BEEN PREPARED FOR SUBMISSION TO THE LOCAL AUTHORITY FOR TOWN & COUNTRY PLANNING AND / OR BUILDING REGULATION PURPOSES ONLY AND DO NOT CONSTITUTE FULL WORKING DRAWINGS. INFORMATION NOTED ON THE PLANS OR ACCOMPANYING DOCUMENTS / DETAILS IS NOT EXHAUSTIVE, AND CONTRACTOR TO CHECK WITH CLIENT AS TO ANY ADDITIONAL WORK NOT SPECIFICALLY NOTED OR IMPLIED.
- ALL MATERIALS ARE TO BE USED IN STRICT ACCORDANCE WITH THE RECOMMENDATIONS OF THE MANUFACTURERS
- ANY WORK COMMENCING ON SITE PRIOR TO BUILDING REGULATIONS APPROVAL IS NOT RECOMMENDED AND IS ENTIRELY THE RESPONSIBILITY OF THE CLIENT.



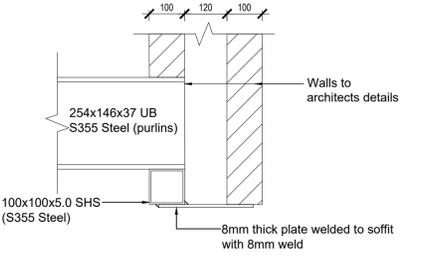
MK2 to MK1 PURLIN BEARING CONNECTION (1:10)



MK5 BEAM & PLATE DETAIL (1:10)



MK6 BEAM & PLATE DETAIL (1:10)



MK7 BEAM & PLATE DETAIL (1:10)



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client: Mr & Mrs Copnall
project: Westwick Lock House, Harrogate

drawing: Proposed Garage Superstructure Plans & Details
status: BUILDING REGULATIONS

job no:	1103	dwg no:	50	rev:	
scale:	1:50@ A1	drawn:	CJM	check:	LA
date:	August 2022	© This drawing is copyright of LARK Architects Ltd			

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Project Details: Westwick Lock House, Boroughbridge
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Superstructure Design

LOADING STREET

ROOF

		Sowl.
gk	• ZINC ROOF	0.1
	• BATTENS + FELT	0.1
	• LAGGERS	0.15
	• SOWELS	0.1
	• PLASTERBOARD	0.15
		<u>0.6</u>
		0.6 w/m^2

gk	• SNOW LOAD	0.6 w/m^2
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EXTERNAL WALL (ALUMINUM)

gk	• 100mm BLOCK + PLASTERBOARD	1.95
	• INSULATION	0.15
	• 100mm BLOCK	1.8
	• FENCER CLADDING	0.3
		<u>4.2</u>
		4.2 w/m^2

EXTERNAL WALL (BRICK)

gk	• 100mm BLOCK + PLASTERBOARD	1.95
	• INSULATION	0.15
	• 100mm BRICK	2.2
		<u>4.3</u>
		4.3 w/m^2

FIRST FLOOR

gk	• BATTENS + JOISTS	0.35
	• SKEFF	0.15
	• SOWELS	0.1
	• PLASTERBOARD	0.15
		<u>0.75</u>
		0.75 w/m^2

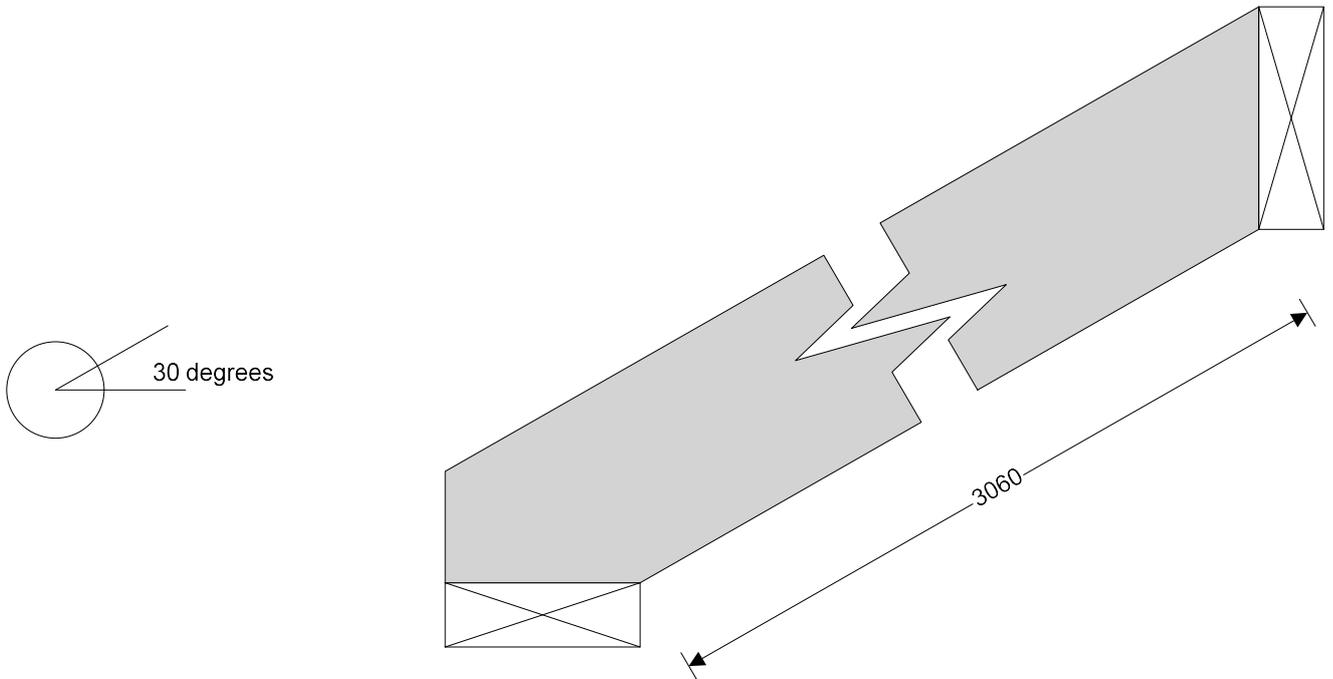
gk	• EXPOSED CONCRETE	1.5
	• PARTITIONS	0.5
		<u>2.0</u>
		2.0 w/m^2

Project Westwick Lock House (Garage)				Job Ref. 1103	
Section Mr & Mrs Copnall				Sheet no./rev. 1	
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TIMBER RAFTER DESIGN (BS5268)

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

Breadth of timber sections;	$b = 50 \text{ mm}$
Depth of timber sections;	$h = 150 \text{ mm}$
Rafter spacing;	$s = 400 \text{ mm}$
Rafter slope;	$\alpha = 30.0 \text{ deg}$
Clear span of rafter on horizontal;	$L_{clh} = 2650 \text{ mm}$
Clear span of rafter on slope;	$L_{cl} = L_{clh} / \cos(\alpha) = 3060 \text{ mm}$
Rafter span;	Single span
Timber strength class;	C16

Section properties

Cross sectional area of rafter;	$A = b \times h = 7500 \text{ mm}^2$
Section modulus;	$Z = b \times h^2 / 6 = 187500 \text{ mm}^3$
Second moment of area;	$I = b \times h^3 / 12 = 14062500 \text{ mm}^4$
Radius of gyration;	$r = \sqrt{I / A} = 43.3 \text{ mm}$

Loading details

Rafter self weight;	$F_j = b \times h \times \rho_{char} \times g_{acc} = 0.02 \text{ kN/m}$
Dead load on slope;	$F_d = 0.60 \text{ kN/m}^2$
Imposed load on plan;	$F_u = 0.60 \text{ kN/m}^2$
Imposed point load;	$F_p = 0.90 \text{ kN}$

Modification factors

Section depth factor;	$K_7 = (300 \text{ mm} / h)^{0.11} = 1.08$
Load sharing factor;	$K_8 = 1.10$

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Consider long term load condition

Load duration factor;	$K_3 = 1.00$
Total UDL perpendicular to rafter;	$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = 0.228 \text{ kN/m}$
Notional bearing length;	$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 3 \text{ mm}$
Effective span;	$L_{eff} = L_{cl} + L_b = 3063 \text{ mm}$

Check bending stress

Bending stress parallel to grain;	$\sigma_m = 5.300 \text{ N/mm}^2$
Permissible bending stress;	$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 6.292 \text{ N/mm}^2$
Applied bending stress;	$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 1.423 \text{ N/mm}^2$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain;	$\sigma_c = 6.800 \text{ N/mm}^2$
Minimum modulus of elasticity;	$E_{min} = 5800 \text{ N/mm}^2$
Compression member factor;	$K_{12} = 0.58$
Permissible compressive stress;	$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 4.324 \text{ N/mm}^2$
Applied compressive stress;	$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.161 \text{ N/mm}^2$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress;	$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 11.441 \text{ N/mm}^2$
Euler coefficient;	$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.988$
Combined axial compression and bending check;	$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.266; < 1$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain;	$\tau = 0.670 \text{ N/mm}^2$
Permissible shear stress;	$\tau_{adm} = \tau \times K_3 \times K_8 = 0.737 \text{ N/mm}^2$
Applied shear stress;	$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = 0.070 \text{ N/mm}^2$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection;	$\delta_{adm} = 0.003 \times L_{eff} = 9.189 \text{ mm}$
Bending deflection;	$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = 2.107 \text{ mm}$
Shear deflection;	$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = 0.078 \text{ mm}$
Total deflection;	$\delta_{max} = \delta_b + \delta_s = 2.185 \text{ mm}$

PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor;	$K_3 = 1.25$
Total UDL perpendicular to rafter;	$F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) = 0.408 \text{ kN/m}$
Notional bearing length;	$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 5 \text{ mm}$
Effective span;	$L_{eff} = L_{cl} + L_b = 3065 \text{ mm}$

Check bending stress

Bending stress parallel to grain;	$\sigma_m = 5.300 \text{ N/mm}^2$
Permissible bending stress;	$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 7.865 \text{ N/mm}^2$
Applied bending stress;	$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 2.553 \text{ N/mm}^2$

PASS - Applied bending stress within permissible limits

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Check compressive stress parallel to grain

Compression stress parallel to grain;	$\sigma_c = 6.800 \text{ N/mm}^2$
Minimum modulus of elasticity;	$E_{min} = 5800 \text{ N/mm}^2$
Compression member factor;	$K_{12} = 0.53$
Permissible compressive stress;	$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 4.988 \text{ N/mm}^2$
Applied compressive stress;	$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.289 \text{ N/mm}^2$
PASS - Applied compressive stress within permissible limits	

Check combined bending and compressive stress parallel to grain

Euler stress;	$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 11.424 \text{ N/mm}^2$
Euler coefficient;	$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.980$
Combined axial compression and bending check;	$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.389; < 1$
PASS - Combined compressive and bending stresses are within permissible limits	

Check shear stress

Shear stress parallel to grain;	$\tau = 0.670 \text{ N/mm}^2$
Permissible shear stress;	$\tau_{adm} = \tau \times K_3 \times K_8 = 0.921 \text{ N/mm}^2$
Applied shear stress;	$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = 0.125 \text{ N/mm}^2$
PASS - Applied shear stress within permissible limits	

Check deflection

Permissible deflection;	$\delta_{adm} = 0.003 \times L_{eff} = 9.195 \text{ mm}$
Bending deflection;	$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = 3.785 \text{ mm}$
Shear deflection;	$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = 0.139 \text{ mm}$
Total deflection;	$\delta_{max} = \delta_b + \delta_s = 3.925 \text{ mm}$
PASS - Total deflection within permissible limits	

Consider short term load condition

Load duration factor;	$K_3 = 1.50$
Total UDL perpendicular to rafter;	$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = 0.228 \text{ kN/m}$
Notional bearing length;	$L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 6 \text{ mm}$
Effective span;	$L_{eff} = L_{cl} + L_b = 3066 \text{ mm}$

Check bending stress

Bending stress parallel to grain;	$\sigma_m = 5.300 \text{ N/mm}^2$
Permissible bending stress;	$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 9.438 \text{ N/mm}^2$
Applied bending stress;	$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) + F_p \times \cos(\alpha) \times L_{eff} / (4 \times Z) = 4.613 \text{ N/mm}^2$
PASS - Applied bending stress within permissible limits	

Check compressive stress parallel to grain

Compression stress parallel to grain;	$\sigma_c = 6.800 \text{ N/mm}^2$
Minimum modulus of elasticity;	$E_{min} = 5800 \text{ N/mm}^2$
Compression member factor;	$K_{12} = 0.49$
Permissible compressive stress;	$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 5.511 \text{ N/mm}^2$
Applied compressive stress;	$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) + F_p \times \sin(\alpha) / A = 0.221 \text{ N/mm}^2$
PASS - Applied compressive stress within permissible limits	

Check combined bending and compressive stress parallel to grain

Euler stress;	$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 11.417 \text{ N/mm}^2$
Euler coefficient;	$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.986$
Combined axial compression and bending check;	$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.536; < 1$

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CJM	24/08/2022				

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain;

$$\tau = 0.670 \text{ N/mm}^2$$

Permissible shear stress;

$$\tau_{adm} = \tau \times K_3 \times K_8 = 1.106 \text{ N/mm}^2$$

Applied shear stress;

$$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A) = 0.226 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection;

$$\delta_{adm} = 0.003 \times L_{eff} = 9.198 \text{ mm}$$

Bending deflection;

$$\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha) / 48) / (E_{mean} \times I) = 5.898 \text{ mm}$$

Shear deflection;

$$\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{mean} \times A) = 0.252 \text{ mm}$$

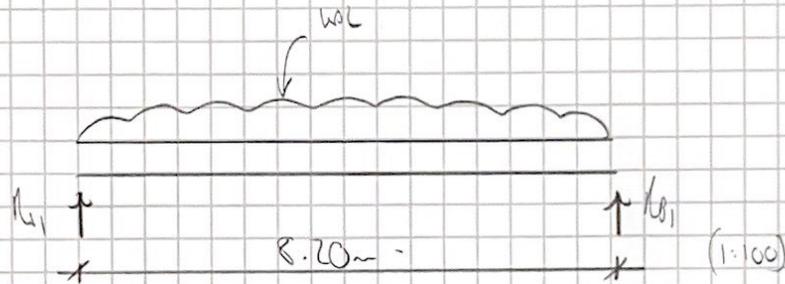
Total deflection;

$$\delta_{max} = \delta_b + \delta_s = 6.150 \text{ mm}$$

PASS - Total deflection within permissible limits

;

Beam MK1 - CLEAR SPAN 8.20m S355 STEEL.



WDL LOADS:

SWL:

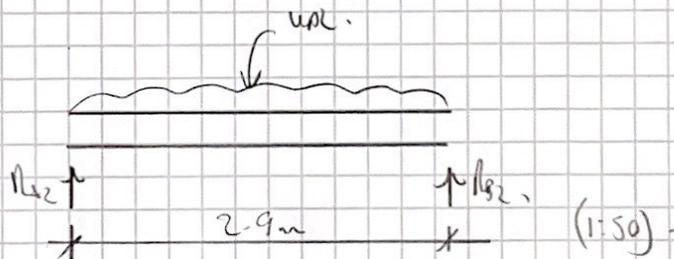
gk · ROOF (0.6m/m²) × LOADED WIDTH 3.6m/2

1.1k/m

qk · ROOF (0.6m/m²) × LOADED WIDTH 3.6m/2

1.1k/m

Beam MK2 - CLEAR SPAN 2.9m S355 STEEL.



WDL LOADS:

SWL:

gk · ROOF (0.6m/m²) × LOADED WIDTH 3.6m/2

1.1k/m

qk · ROOF (0.6m/m²) × LOADED WIDTH 3.6m/2

1.1k/m

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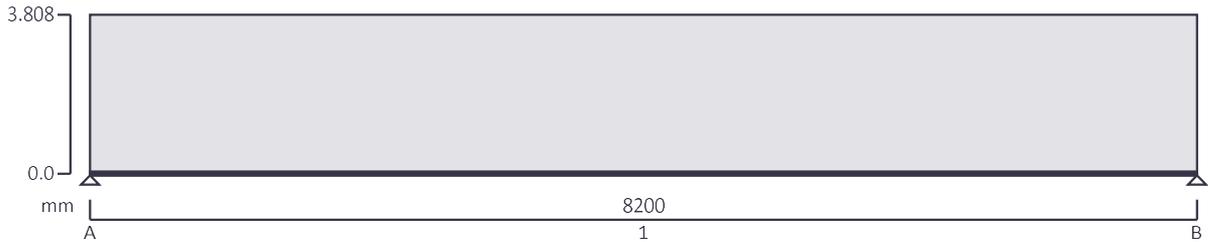
MK1 STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

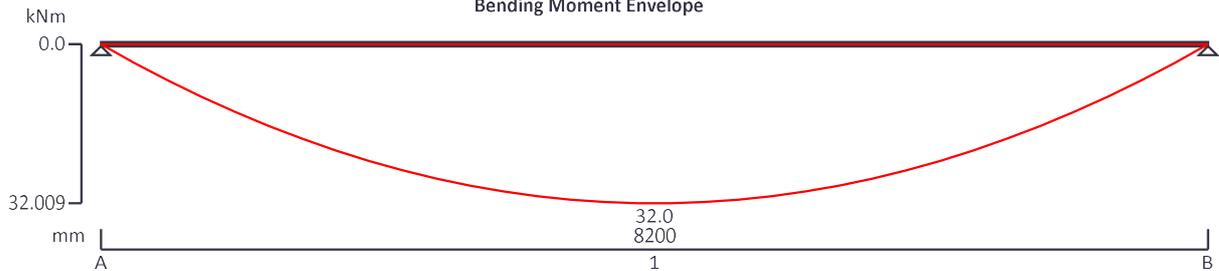
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

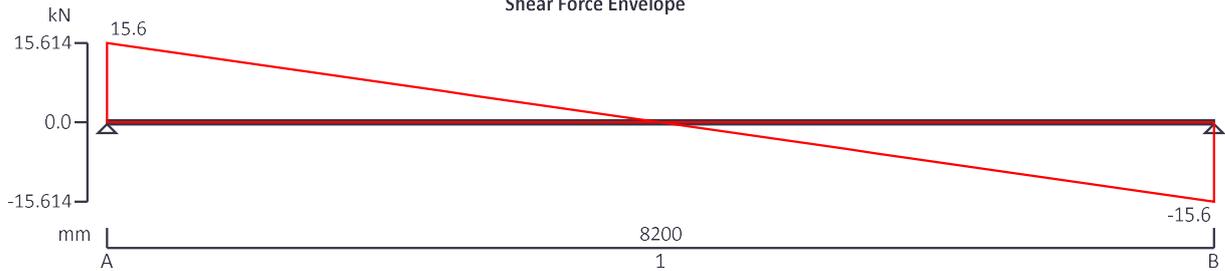
Load Envelope - Combination 1



Bending Moment Envelope



Shear Force Envelope



Support conditions

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

Applied loading

Beam loads	Dead full UDL 1.1 kN/m
	Imposed full UDL 1.1 kN/m
	Dead self weight of beam $\times 1$

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Load combinations

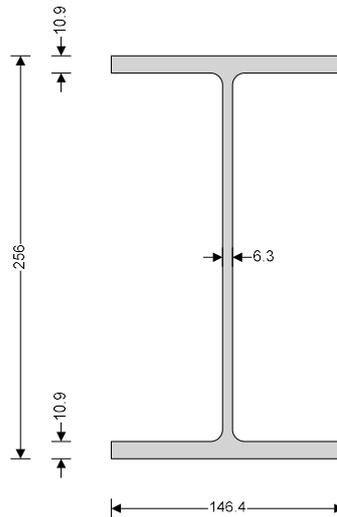
Load combination 1	Support A	Dead × 1.40 Imposed × 1.60
	Support B	Dead × 1.40 Imposed × 1.60

Analysis results

Maximum moment;	$M_{max} = 32 \text{ kNm};$	$M_{min} = 0 \text{ kNm}$
Maximum shear;	$V_{max} = 15.6 \text{ kN};$	$V_{min} = -15.6 \text{ kN}$
Deflection;	$\delta_{max} = 13.3 \text{ mm};$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A;	$R_{A_max} = 15.6 \text{ kN};$	$R_{A_min} = 15.6 \text{ kN}$
Unfactored dead load reaction at support A;	$R_{A_Dead} = 6 \text{ kN}$	
Unfactored imposed load reaction at support A;	$R_{A_Imposed} = 4.5 \text{ kN}$	
Maximum reaction at support B;	$R_{B_max} = 15.6 \text{ kN};$	$R_{B_min} = 15.6 \text{ kN}$
Unfactored dead load reaction at support B;	$R_{B_Dead} = 6 \text{ kN}$	
Unfactored imposed load reaction at support B;	$R_{B_Imposed} = 4.5 \text{ kN}$	

Section details

Section type;	UKB 254x146x37 (Tata Steel Advance)
Steel grade;	S355
From table 9: Design strength p_y	
Thickness of element;	$\max(T, t) = 10.9 \text{ mm}$
Design strength;	$p_y = 355 \text{ N/mm}^2$
Modulus of elasticity;	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports only

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;	$K_{LT,A} = 1.00;$
	$K_{LT,B} = 1.00;$

Project Westwick Lock House (Garage)				Job Ref. 1103	
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Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section; $d = \mathbf{219 \text{ mm}}$
 $d / t = 39.5 \times \varepsilon \leq 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = \mathbf{73.2 \text{ mm}}$
 $b / T = 7.6 \times \varepsilon \leq 9 \times \varepsilon$; Class 1 plastic
Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{15.6 \text{ kN}}$
 $d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \times D = \mathbf{1613 \text{ mm}^2}$
 Design shear resistance; $P_v = 0.6 \times p_y \times A_v = \mathbf{343.5 \text{ kN}}$
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1_{\max}}), \text{abs}(M_{s1_{\min}})) = \mathbf{32 \text{ kNm}}$
 Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{171.5 \text{ kNm}}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} = \mathbf{8200 \text{ mm}}$
 Slenderness ratio; $\lambda = L_E / r_{yy} = \mathbf{235.745}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = \mathbf{0.890}$
 Torsional index; $x = \mathbf{24.332}$
 Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.647}$
 Ratio - cl.4.3.6.9; $\beta_w = \mathbf{1.000}$
 Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_w]} = \mathbf{135.798}$
 Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{30.198}$
 $\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = \mathbf{7.0}$
 Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.739}$
 Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{109.7 \text{ N/mm}^2}$
 $\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{272.9 \text{ N/mm}^2}$
 Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{84.4 \text{ N/mm}^2}$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = \mathbf{24 \text{ kNm}}$
 Moment at centre-line of segment; $M_3 = \mathbf{32 \text{ kNm}}$
 Moment at three quarter point of segment; $M_4 = \mathbf{24 \text{ kNm}}$
 Maximum moment in segment; $M_{\text{abs}} = \mathbf{32 \text{ kNm}}$
 Maximum moment governing buckling resistance; $M_{LT} = M_{\text{abs}} = \mathbf{32 \text{ kNm}}$
 Equivalent uniform moment factor for lateral-torsional buckling;
 $m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{\text{abs}}, 0.44) = \mathbf{0.925}$

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Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment;

$$M_b = p_b \times S_{xx} = 40.8 \text{ kNm}$$

$$M_b / m_{LT} = 44.1 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment**Check vertical deflection - Section 2.5.2**

Consider deflection due to dead and imposed loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 360 = 22.778 \text{ mm}$$

Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 13.294 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

;

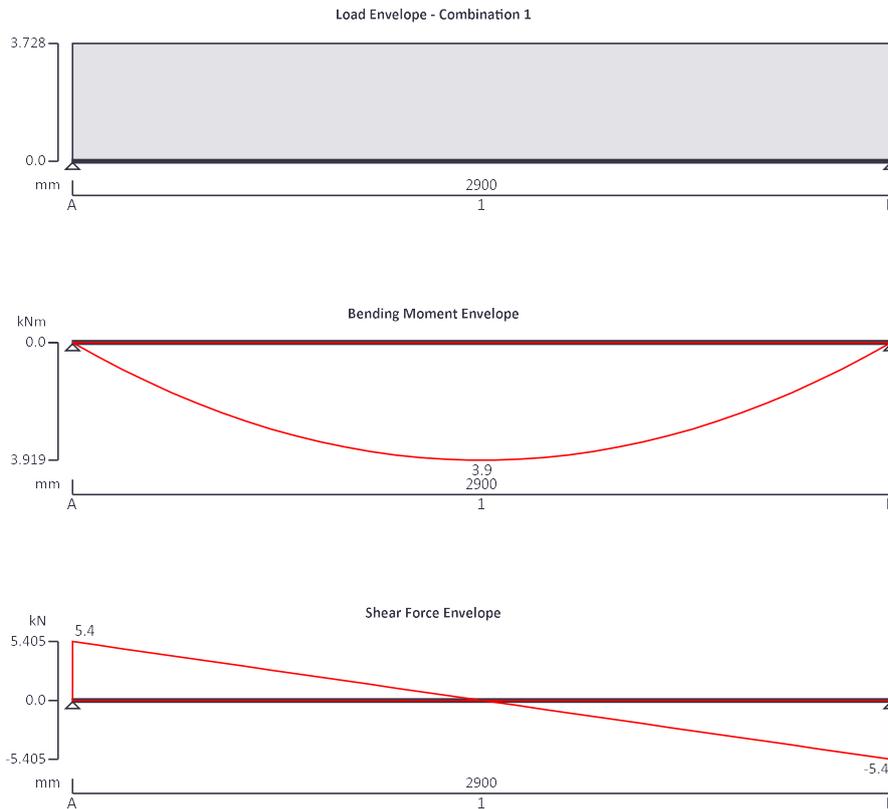
Project Westwick Lock House (Garage)				Job Ref. 1103	
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MK2 STEEL BEAM ANALYSIS & DESIGN (BS5950)

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 full UDL 1.1 kN/m Imposed full UDL 1.1 kN/m Dead self weight of beam \times 1
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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

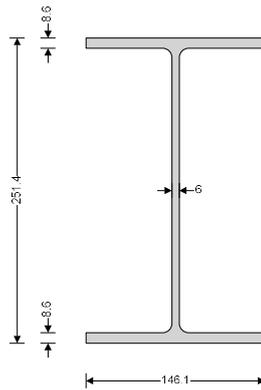
Maximum moment;	$M_{max} = 3.9$ kNm;	$M_{min} = 0$ kNm
Maximum shear;	$V_{max} = 5.4$ kN;	$V_{min} = -5.4$ kN
Deflection;	$\delta_{max} = 0.3$ mm;	$\delta_{min} = 0$ mm
Maximum reaction at support A;	$R_{A_{max}} = 5.4$ kN;	$R_{A_{min}} = 5.4$ kN
Unfactored dead load reaction at support A;	$R_{A_{Dead}} = 2$ kN	
Unfactored imposed load reaction at support A;	$R_{A_{Imposed}} = 1.6$ kN	
Maximum reaction at support B;	$R_{B_{max}} = 5.4$ kN;	$R_{B_{min}} = 5.4$ kN
Unfactored dead load reaction at support B;	$R_{B_{Dead}} = 2$ kN	
Unfactored imposed load reaction at support B;	$R_{B_{Imposed}} = 1.6$ kN	

Section details

Section type; **UKB 254x146x31 (Tata Steel Advance)**
Steel grade; **S355**

From table 9: Design strength p_y

Thickness of element; $\max(T, t) = 8.6$ mm
Design strength; $p_y = 355$ N/mm²
Modulus of elasticity; $E = 205000$ N/mm²



Lateral restraint

Span 1 has lateral restraint at supports only

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; $K_{LTA} = 1.00$;
 $K_{LTB} = 1.00$;

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; $d = 219$ mm
 $d / t = 41.5 \times \varepsilon \leq 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = 73.1$ mm
 $b / T = 9.7 \times \varepsilon \leq 10 \times \varepsilon$; Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = 5.4$ kN

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

Web does not need to be checked for shear buckling

Shear area;

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

Design shear resistance;

$$P_v = 0.6 \times p_y \times A_v = \mathbf{321.3 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment;

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{3.9 \text{ kNm}}$$

Moment capacity low shear - cl.4.2.5.2;

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{139.5 \text{ kNm}}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling;

$$L_E = 1.0 \times L_{s1} = \mathbf{2900 \text{ mm}}$$

Slenderness ratio;

$$\lambda = L_E / r_{yy} = \mathbf{86.349}$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter;

$$u = \mathbf{0.880}$$

Torsional index;

$$x = \mathbf{29.601}$$

Slenderness factor;

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.915}$$

Ratio - cl.4.3.6.9;

$$\beta_w = \mathbf{1.000}$$

Equivalent slenderness - cl.4.3.6.7;

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_w]} = \mathbf{69.504}$$

Limiting slenderness - Annex B.2.2;

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{30.198}$$

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant;

$$\alpha_{LT} = \mathbf{7.0}$$

Perry factor;

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.275}$$

Euler stress;

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{418.8 \text{ N/mm}^2}$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{444.5 \text{ N/mm}^2}$$

Bending strength - Annex B.2.1;

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{223.3 \text{ N/mm}^2}$$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment;

$$M_2 = \mathbf{2.9 \text{ kNm}}$$

Moment at centre-line of segment;

$$M_3 = \mathbf{3.9 \text{ kNm}}$$

Moment at three quarter point of segment;

$$M_4 = \mathbf{2.9 \text{ kNm}}$$

Maximum moment in segment;

$$M_{abs} = \mathbf{3.9 \text{ kNm}}$$

Maximum moment governing buckling resistance;

$$M_{LT} = M_{abs} = \mathbf{3.9 \text{ kNm}}$$

Equivalent uniform moment factor for lateral-torsional buckling;

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.925}$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment;

$$M_b = p_b \times S_{xx} = \mathbf{87.8 \text{ kNm}}$$

$$M_b / m_{LT} = \mathbf{94.9 \text{ kNm}}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 360 = \mathbf{8.056 \text{ mm}}$$

Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{0.255 \text{ mm}}$$

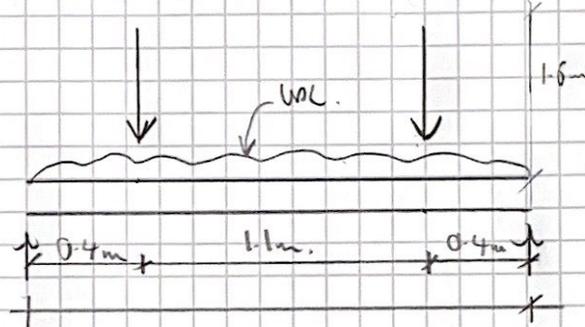
PASS - Maximum deflection does not exceed deflection limit

;

Beam MK3 - CURVE SPAN 1.9m S355 STEEL.

MK1 + MK2 MK2 MK1 + MK2
 $g_k = 6.0w$ $g_k = 2.0w$ " "
 $q_k = 4.5w$ $q_k = 1.6w$ " "

WAL CUR SPREAD
 TAKE CROSS AREA
 LENGTH OF BEAM.



WAL LOADS:

SECV:

g_k • 100mm BLOCK + POLYSTYRENE (1.95w/m²) x HEIGHT, 1.6m
 • $g_k = 16.0w / 1.6m$

3.1w/m
 10.0w/m

13.1w/m

q_k • $q_k = 12.2w / 1.6m$

7.6w/m

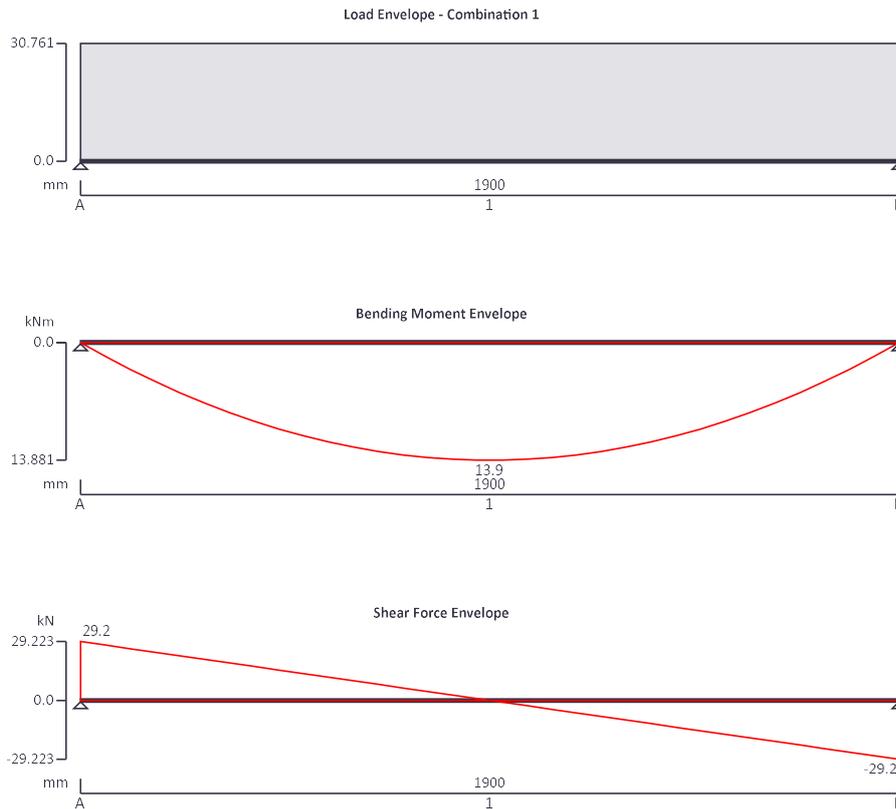
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MK3 STEEL BEAM ANALYSIS & DESIGN (BS5950)

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	Imposed full UDL 7.6 kN/m Dead full UDL 13.1 kN/m Dead self weight of beam $\times 1$
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Load combinations

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

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Analysis results

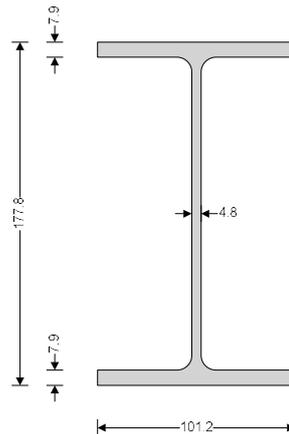
Maximum moment;	$M_{max} = 13.9 \text{ kNm};$	$M_{min} = 0 \text{ kNm}$
Maximum shear;	$V_{max} = 29.2 \text{ kN};$	$V_{min} = -29.2 \text{ kN}$
Deflection;	$\delta_{max} = 1.3 \text{ mm};$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A;	$R_{A_max} = 29.2 \text{ kN};$	$R_{A_min} = 29.2 \text{ kN}$
Unfactored dead load reaction at support A;	$R_{A_Dead} = 12.6 \text{ kN}$	
Unfactored imposed load reaction at support A;	$R_{A_Imposed} = 7.2 \text{ kN}$	
Maximum reaction at support B;	$R_{B_max} = 29.2 \text{ kN};$	$R_{B_min} = 29.2 \text{ kN}$
Unfactored dead load reaction at support B;	$R_{B_Dead} = 12.6 \text{ kN}$	
Unfactored imposed load reaction at support B;	$R_{B_Imposed} = 7.2 \text{ kN}$	

Section details

Section type; **UKB 178x102x19 (Tata Steel Advance)**
Steel grade; **S355**

From table 9: Design strength p_y

Thickness of element; $\max(T, t) = 7.9 \text{ mm}$
Design strength; $p_y = 355 \text{ N/mm}^2$
Modulus of elasticity; $E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports only

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

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

Internal compression parts - Table 11

Depth of section; $d = 146.8 \text{ mm}$
 $d / t = 34.7 \times \epsilon \leq 80 \times \epsilon;$ Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = 50.6 \text{ mm}$
 $b / T = 7.3 \times \epsilon \leq 9 \times \epsilon;$ Class 1 plastic

Section is class 1 plastic

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Shear capacity - Section 4.2.3

Design shear force;

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 29.2 \text{ kN}$$

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

Web does not need to be checked for shear buckling

Shear area;

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

Design shear resistance;

$$P_v = 0.6 \times p_y \times A_v = 181.8 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment;

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 13.9 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2;

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 60.8 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling;

$$L_E = 1.0 \times L_{s1} = 1900 \text{ mm}$$

Slenderness ratio;

$$\lambda = L_E / r_{yy} = 80.042$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter;

$$u = 0.888$$

Torsional index;

$$x = 22.560$$

Slenderness factor;

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.885$$

Ratio - cl.4.3.6.9;

$$\beta_W = 1.000$$

Equivalent slenderness - cl.4.3.6.7;

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} = 62.886$$

Limiting slenderness - Annex B.2.2;

$$\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 30.198$$

$\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant;

$$\alpha_{LT} = 7.0$$

Perry factor;

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.229$$

Euler stress;

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 511.6 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 491.8 \text{ N/mm}^2$$

Bending strength - Annex B.2.1;

$$p_b = p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 246.3 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment;

$$M_2 = 10.4 \text{ kNm}$$

Moment at centre-line of segment;

$$M_3 = 13.9 \text{ kNm}$$

Moment at three quarter point of segment;

$$M_4 = 10.4 \text{ kNm}$$

Maximum moment in segment;

$$M_{abs} = 13.9 \text{ kNm}$$

Maximum moment governing buckling resistance;

$$M_{LT} = M_{abs} = 13.9 \text{ kNm}$$

Equivalent uniform moment factor for lateral-torsional buckling;

$$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = 0.925$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment;

$$M_b = p_b \times S_{xx} = 42.2 \text{ kNm}$$

$$M_b / m_{LT} = 45.6 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 360 = 5.278 \text{ mm}$$

Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 1.275 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

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TIMBER JOIST DESIGN (BS5268)

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

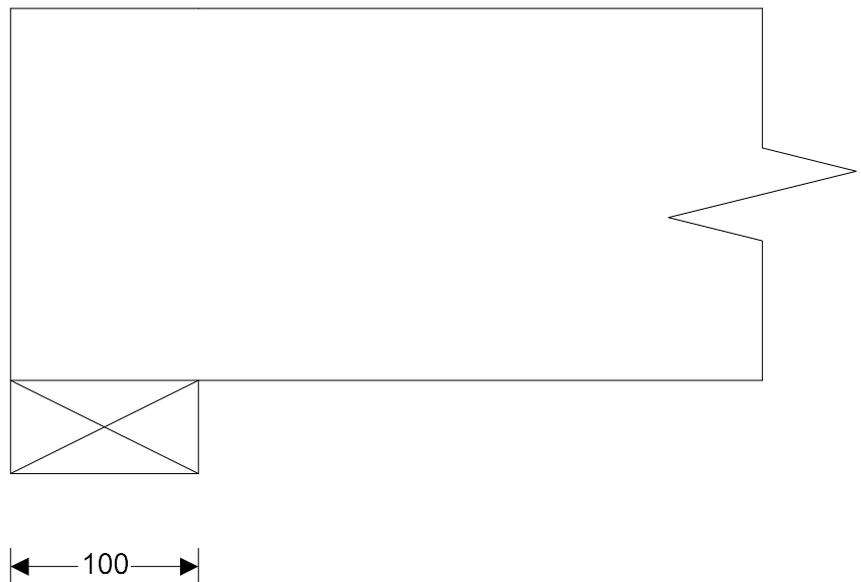
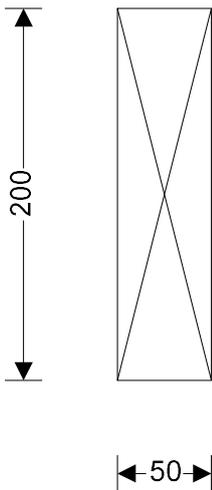
Joist details

Joist breadth; **b = 50 mm**
 Joist depth; **h = 200 mm**
 Joist spacing; **s = 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} = 4100 mm**



Section properties

Second moment of area; **I = b × h³ / 12 = 33333333 mm⁴**

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Section modulus; $Z = b \times h^2 / 6 = 333333 \text{ mm}^3$

Loading details

Joist self weight; $F_{swt} = b \times h \times \rho_{char} \times g_{acc} = 0.03 \text{ kN/m}$
 Dead load; $F_{d_udl} = 0.75 \text{ kN/m}^2$
 Imposed UDL(Long term); $F_{i_udl} = 2.00 \text{ kN/m}^2$
 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 $K_{2e} = 1.00$
 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; $M = 2.383 \text{ kNm}$
 Maximum shear force; $V = 2.325 \text{ kN}$
 Maximum support reaction; $R = 2.325 \text{ kN}$
 Maximum deflection; $\delta = 12.017 \text{ mm}$

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; $\sigma_{m_max} = M / Z = 7.150 \text{ N/mm}^2$
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; $\tau_{max} = 3 \times V / (2 \times b \times h) = 0.349 \text{ N/mm}^2$
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; $\sigma_{c_max} = R / (b \times L_b) = 0.465 \text{ N/mm}^2$
PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection; $\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 12.300 \text{ mm}$
 Bending deflection (based on E_{mean}); $\delta_{bending} = 11.593 \text{ mm}$
 Shear deflection; $\delta_{shear} = 0.424 \text{ mm}$
 Total deflection; $\delta = \delta_{bending} + \delta_{shear} = 12.017 \text{ mm}$
PASS - Actual deflection within permissible limits

Consider medium term loads

Load duration factor; $K_3 = 1.25$
 Maximum bending moment; $M = 2.137 \text{ kNm}$
 Maximum shear force; $V = 2.085 \text{ kN}$
 Maximum support reaction; $R = 2.085 \text{ kN}$

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Maximum deflection;

$$\delta = 9.381 \text{ mm}$$

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;

$$\sigma_{m_max} = M / Z = 6.412 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress;

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

Permissible shear stress;

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

Applied shear stress;

$$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.313 \text{ N/mm}^2$$

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;

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

Applied bearing stress;

$$\sigma_{c_max} = R / (b \times L_b) = 0.417 \text{ N/mm}^2$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection;

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 12.300 \text{ mm}$$

Bending deflection (based on E_{mean});

$$\delta_{bending} = 9.001 \text{ mm}$$

Shear deflection;

$$\delta_{shear} = 0.380 \text{ mm}$$

Total deflection;

$$\delta = \delta_{bending} + \delta_{shear} = 9.381 \text{ mm}$$

PASS - Actual deflection within permissible limits

;



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Job no. :

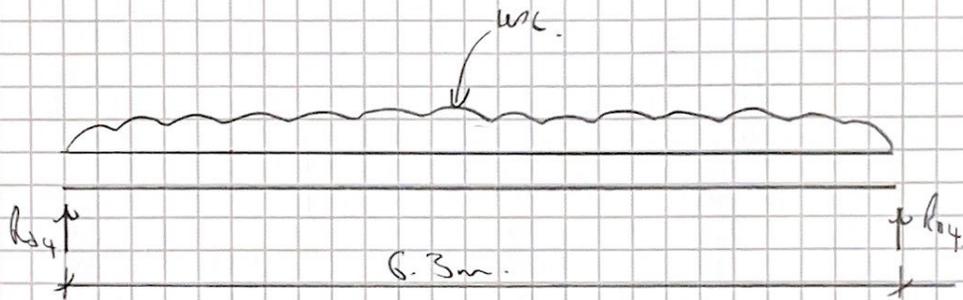
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Beam MK 4 - CURVE SPAN 6.3m S355 STEEL



WOL LOADS:

SEW:

gk • Floor Floor (0.75 w/m²) x COVERED WIDTH 8.2m / 2

3.1 w/m²

gk • Floor Floor (2.0 w/m²) x COVERED WIDTH 8.2m / 2

8.2 w/m²

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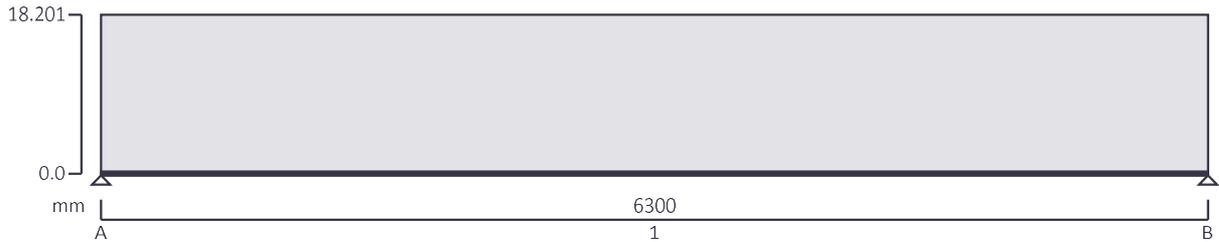
MK4 STEEL BEAM ANALYSIS & DESIGN (BS5950)

STEEL BEAM ANALYSIS & DESIGN (BS5950)

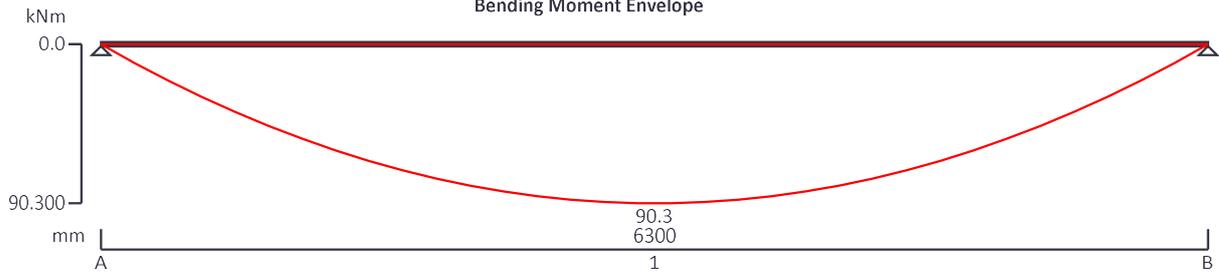
In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07

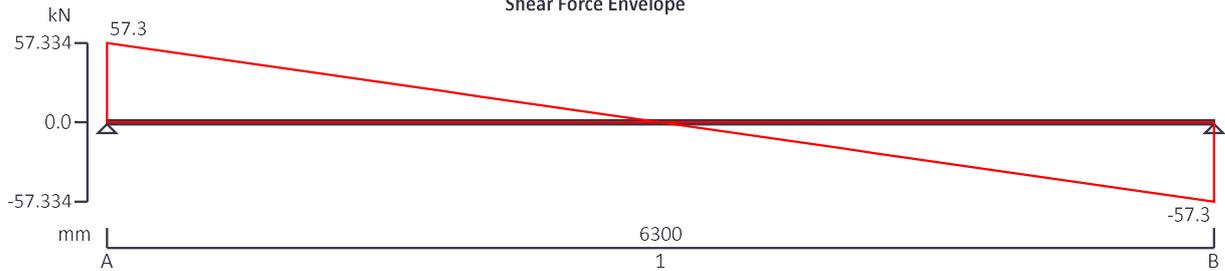
Load Envelope - Combination 1



Bending Moment Envelope



Shear Force Envelope



Support conditions

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

Applied loading

Beam loads	Dead full UDL 3.1 kN/m
	Imposed full UDL 8.2 kN/m
	Dead self weight of beam $\times 1$

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Load combinations

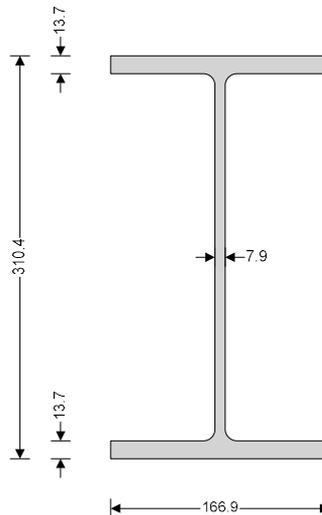
Load combination 1	Support A	Dead × 1.40 Imposed × 1.60
	Support B	Dead × 1.40 Imposed × 1.60

Analysis results

Maximum moment;	$M_{max} = 90.3 \text{ kNm};$	$M_{min} = 0 \text{ kNm}$
Maximum shear;	$V_{max} = 57.3 \text{ kN};$	$V_{min} = -57.3 \text{ kN}$
Deflection;	$\delta_{max} = 10.1 \text{ mm};$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A;	$R_{A_max} = 57.3 \text{ kN};$	$R_{A_min} = 57.3 \text{ kN}$
Unfactored dead load reaction at support A;	$R_{A_Dead} = 11.4 \text{ kN}$	
Unfactored imposed load reaction at support A;	$R_{A_Imposed} = 25.8 \text{ kN}$	
Maximum reaction at support B;	$R_{B_max} = 57.3 \text{ kN};$	$R_{B_min} = 57.3 \text{ kN}$
Unfactored dead load reaction at support B;	$R_{B_Dead} = 11.4 \text{ kN}$	
Unfactored imposed load reaction at support B;	$R_{B_Imposed} = 25.8 \text{ kN}$	

Section details

Section type;	UKB 305x165x54 (Tata Steel Advance)
Steel grade;	S355
From table 9: Design strength p_y	
Thickness of element;	$\max(T, t) = 13.7 \text{ mm}$
Design strength;	$p_y = 355 \text{ N/mm}^2$
Modulus of elasticity;	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has lateral restraint at supports only

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;	$K_{LT,A} = 1.00;$
	$K_{LT,B} = 1.00;$

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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; $d = 265.2 \text{ mm}$
 $d / t = 38.1 \times \varepsilon \leq 80 \times \varepsilon$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = 83.5 \text{ mm}$
 $b / T = 6.9 \times \varepsilon \leq 9 \times \varepsilon$; Class 1 plastic
Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 57.3 \text{ kN}$
 $d / t < 70 \times \varepsilon$
Web does not need to be checked for shear buckling
 Shear area; $A_v = t \times D = 2452 \text{ mm}^2$
 Design shear resistance; $P_v = 0.6 \times p_y \times A_v = 522.3 \text{ kN}$
PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1_{\max}}), \text{abs}(M_{s1_{\min}})) = 90.3 \text{ kNm}$
 Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 300.4 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.0 \times L_{s1} = 6300 \text{ mm}$
 Slenderness ratio; $\lambda = L_E / r_{yy} = 160.241$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = 0.889$
 Torsional index; $x = 23.612$
 Slenderness factor; $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.742$
 Ratio - cl.4.3.6.9; $\beta_w = 1.000$
 Equivalent slenderness - cl.4.3.6.7; $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_w]} = 105.701$
 Limiting slenderness - Annex B.2.2; $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = 30.198$
 $\lambda_{LT} > \lambda_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $\alpha_{LT} = 7.0$
 Perry factor; $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.529$
 Euler stress; $p_E = \pi^2 \times E / \lambda_{LT}^2 = 181.1 \text{ N/mm}^2$
 $\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 315.9 \text{ N/mm}^2$
 Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 127.5 \text{ N/mm}^2$

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment; $M_2 = 67.7 \text{ kNm}$
 Moment at centre-line of segment; $M_3 = 90.3 \text{ kNm}$
 Moment at three quarter point of segment; $M_4 = 67.7 \text{ kNm}$
 Maximum moment in segment; $M_{\text{abs}} = 90.3 \text{ kNm}$
 Maximum moment governing buckling resistance; $M_{LT} = M_{\text{abs}} = 90.3 \text{ kNm}$
 Equivalent uniform moment factor for lateral-torsional buckling;
 $m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{\text{abs}}, 0.44) = 0.925$

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Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment;

$$M_b = p_b \times S_{xx} = 107.8 \text{ kNm}$$

$$M_b / m_{LT} = 116.6 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment**Check vertical deflection - Section 2.5.2**

Consider deflection due to dead and imposed loads

Limiting deflection;

$$\delta_{lim} = L_{s1} / 360 = 17.5 \text{ mm}$$

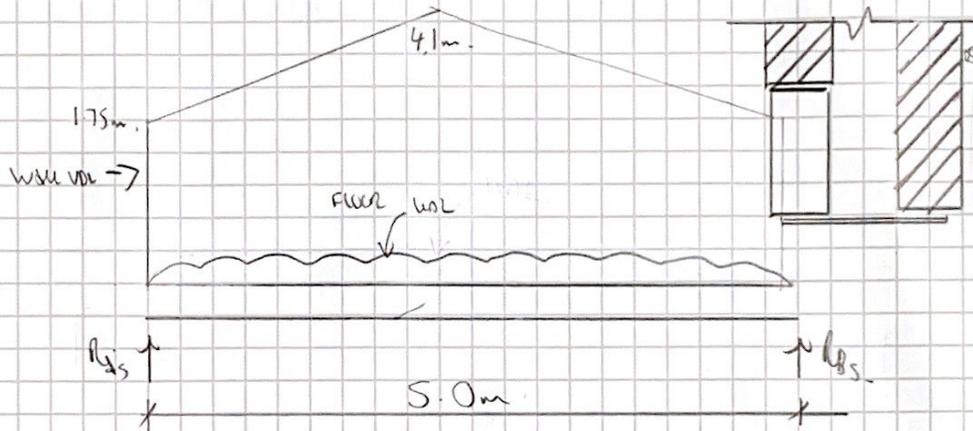
Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 10.12 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

;

MKS - Beam + Plate - Clear span 5.0m S355 Steel



WALL CONCRET. WALL

- | | | |
|----|---|---------------------|
| gh | • FIRST FLOOR (0.75 m ²) x CONCRET WIDTH 4.1m / 2 | 1.55 m ³ |
| gh | • FIRST FLOOR (2.0 m ²) x CONCRET WIDTH 4.1m / 2 | 4.1 m ³ |

WALL CONCR. ROOF

- | | | |
|----|--|--------------------|
| gh | • 100mm BLOCK + PLASTERBOARD (1.95 m ²) x HEIGHT 1.75m | 3.4 m ³ |
| gh | • 100mm BLOCK + PLASTERBOARD (1.95 m ²) x HEIGHT 4.1m | 8.0 m ³ |

WALL CONCR. CEILING

- | | | |
|----|---|--------------------|
| gh | • 100mm BLOCK + CLADDING (2.1 m ²) x HEIGHT 1.75m | 3.6 m ³ |
| gh | • 100mm BLOCK + CLADDING (2.1 m ²) HEIGHT 4.1m | 8.6 m ³ |

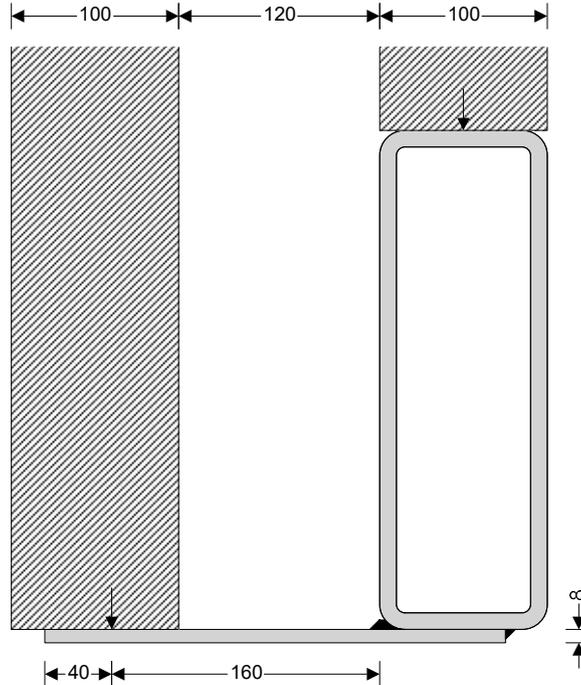
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MK5 STEEL MASONRY SUPPORT (BS5950)

STEEL MASONRY SUPPORT

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

Tedds calculation version 1.0.04



Steel member details

Torsion beam;	RHS 300x100x10.0
Masonry support plate;	User
Steel grade of support plate;	S355
Design strength of support plate;	$p_{ysb} = 355 \text{ N/mm}^2$
Modulus of elasticity;	$E = 205000 \text{ N/mm}^2$
Constant;	$\epsilon = \sqrt{(275 \text{ N/mm}^2 / p_{ysb})} = 0.880$
Length of plate beyond beam;	$l_h = 200 \text{ mm}$
Total length of plate;	$l_{plate} = 275 \text{ mm}$
Thickness of plate;	$t_{sb} = 8 \text{ mm}$
Width of main beam;	$B_{mb} = 100 \text{ mm}$
Area of plate;	$A_{sbu} = t_{sb} \times l_{plate} = 2200.0 \text{ mm}^2$
Distance from weld position to CoG;	$c_{yysb} = l_h / 2 - (l_{plate} - l_h) / 2 = 63 \text{ mm}$

Supported materials detail

Density of masonry on main beam;	$\rho_{m,mb} = 20.0 \text{ kN/m}^3$
Width of masonry on main beam;	$b_{mmb} = 100 \text{ mm}$
Height of masonry on main beam;	$h_{mmb} = 2900 \text{ mm}$
Eccentricity of main beam material;	$e_{mb} = 0 \text{ mm}$
Add dead force main beam (not from masonry);	$P_{Gaddmb} = 1.6 \text{ kN/m}$
Add live force main beam (not from masonry);	$P_{Qaddmb} = 4.1 \text{ kN/m}$
Density of masonry on support beam;	$\rho_{m,sb} = 22.0 \text{ kN/m}^3$
Width of masonry on support beam;	$b_{msb} = 100 \text{ mm}$

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Height of masonry on support beam; $h_{msb} = 2900 \text{ mm}$
 Add dead force support beam (not from masonry); $P_{Gaddsb} = 0.0 \text{ kN/m}$
 Add live force support beam (not from masonry); $P_{Qaddsb} = 0.0 \text{ kN/m}$

Geometry

Cavity width; $c = 120 \text{ mm}$
 Supported width of masonry; $d_m = l_h + e_{mb} - c = 80 \text{ mm}$

Biaxial stress effects in the plate (SCI-P-110)

Maximum overall bending moment; $M_x = 83.9 \text{ kNm}$
 Dist to NA combined section (CoG torsion beam); $y_{e,all} = (D_{mb} + t_{sb}) \times A_{sbu} / (2 \times (A_{mb} + A_{sbu})) = 35 \text{ mm}$
 Second moment of area of combined section; $I_{xx,all} = (I_{xx,mb} + A_{mb} \times y_{e,all}^2) + A_{sbu} \times (D_{mb} / 2 + t_{sb} / 2 - y_{e,all})^2 = 11646 \text{ cm}^4$
 Elastic section modulus of combined section; $Z_{xx,all} = I_{xx,all} / (D_{mb} / 2 + t_{sb} - y_{e,all}) = 946.48 \text{ cm}^3$
 Section modulus of plate; $Z_{xx,plate} = 1m \times t_{sb}^2 / (6 \times 1m) = 10.67 \text{ cm}^3/m$
 Eccentricity of support beam masonry; $e_1 = 160 \text{ mm}$
 Force of masonry on support plate; $P_1 = (b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ} = 8.9 \text{ kN/m}$
 Bending at heel; $M_{x,plate} = P_1 \times e_1 = 1.4 \text{ kNm/m}$
 Moment capacity of plate; $M_c = 1.2 \times Z_{x,plate} \times p_{ysb} = 4.5 \text{ kNm/m}$

PASS - Design strength exceeds stress at heel

Longitudinal stress due to overall bending; $\sigma_1 = M_x / Z_{xx,all} = 88.6 \text{ N/mm}^2$
 Constant relating to Von Mises curve; $C_{fp} = (4 \times p_{ysb}^2 - 3 \times \sigma_1^2)^{0.5} = 693.2 \text{ N/mm}^2$
 Transverse bending stress ratio limit; $\alpha_{ts} = (C_{fp}^2 - \sigma_1^2) / (2 \times C_{fp} \times p_{ysb}) = 0.960$
 Transverse bending stress ratio; $\alpha_{ts} = M_{x,plate} / M_c = 0.315$

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfactored force on support angle; $P_{1SLS} = b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb} = 6.4 \text{ kN/m}$
 Distance from weld to load position; $a_m = e_1 = 160 \text{ mm}$
 Length of load resultant to edge of plate; $b_m = l_h - e_1 = 40 \text{ mm}$
 Dist from weld to load position as ratio of length; $a_l = a_m / (a_m + b_m) = 0.800$
 Effective second moment of inertia; $I_{eff_def} = t_{sb}^3 / 12 = 42667 \text{ mm}^4/m$
 Deflection at toe; $\delta = (a_l^2 \times (3 - a_l) / 6) \times (P_{1SLS} \times (a_m + b_m)^3) / (E_{S5950} \times I_{eff_def}) = 1.37 \text{ mm}$
 Deflection limit; $\delta_{lim} = 1.67 \text{ mm}$

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Leg length of weld; $s_{weld} = 6 \text{ mm}$
 Throat size of weld; $a_{weld} = 1/\sqrt{2} \times s_{weld} = 4.2 \text{ mm}$
 Shear force at weld position; $R_A = P_1 \times \max((1 + (3 \times e_1) / (2 \times B_{mb}) / 2)), 1.4) = 51.8 \text{ kN/m}$
 Maximum possible force in plate; $R_p = (l_h + B_{mb}) \times t_{sb} \times p_{ysb} = 852.0 \text{ kN}$
 Longitudinal shear between beam and plate; $R_l = 2 \times R_p / L = 340.8 \text{ kN/m}$
 Horizontal shear between beam and plate; $R_h = P_1 \times e_1 / (s_{weld} / 2 + t_{sb} / 2) = 204.2 \text{ kN/m}$
 Resultant weld force; $R_{weld} = (R_A^2 + R_l^2 + R_h^2)^{0.5} = 0.401 \text{ kN/mm}$
 Strength of weld (Table 37); $p_{weld} = 220.0 \text{ N/mm}^2$
 Capacity of full length weld; $p_{c,weld} = a_{weld} \times p_{weld} = 0.933 \text{ kN/mm}$

PASS - Capacity of weld exceeds resultant force on weld

Torsional loading ULS

Loading of support beam masonry; $w_{1ULS} = (h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ} = 8.93 \text{ kN/m}$

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Loading of main beam masonry; $W_{2ULS} = (h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb}) \times \gamma_{fG} + P_{Qaddmb} \times \gamma_{fQ} = \mathbf{16.85}$
kN/m

Self weight of support beam; $W_{3ULS} = A_{sbu} \times \rho_{sb} \times \gamma_{fG} = \mathbf{0.24}$ kN/m

Torsional loading SLS

Loading of support beam masonry; $W_{1SLS} = h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb} = \mathbf{6.38}$ kN/m

Loading of main beam masonry; $W_{2SLS} = h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb} + P_{Qaddmb} = \mathbf{11.45}$ kN/m

Self weight of support beam; $W_{3SLS} = A_{sbu} \times \rho_{sb} = \mathbf{0.17}$ kN/m

Eccentricities

Distance to shear centre of main beam; $e_{0mb} = \mathbf{0}$ mm

Eccentricity of support beam masonry; $e_{1mb} = (B_{mb} + b_{msb}) / 2 + c - e_{mb} = \mathbf{220}$ mm

Eccentricity of main beam masonry; $e_{2mb} = (B_{mb} - b_{mmb}) / 2 - e_{mb} = \mathbf{0}$ mm

Eccentricity of support beam; $e_{3mb} = B_{mb} / 2 + c_{yysb} = \mathbf{113}$ mm

Torsional effects

Applied torque (ULS); $T_{qULS} = \text{abs}(W_{1ULS} \times e_{1mb} + W_{2ULS} \times e_{2mb} + W_{3ULS} \times e_{3mb}) = \mathbf{1.99}$ kNm/m

Total torque (ULS); $T_q = T_{qULS} \times L = \mathbf{9.96}$ kNm

Applied torque (SLS); $T_{qSLS} = \text{abs}(W_{1SLS} \times e_{1mb} + W_{2SLS} \times e_{2mb} + W_{3SLS} \times e_{3mb}) = \mathbf{1.42}$ kNm/m

Total torque (SLS); $T_{qu} = T_{qSLS} \times L = \mathbf{7.12}$ kNm

STEEL BEAM TORSION DESIGN

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

Tedds calculation version 2.0.02

Section details

Section type; RHS 300x100x10.0

Steel grade; S355

Design strength; $p_{yw} = p_y = \mathbf{355}$ N/mm²

Constant; $\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_y)} = \mathbf{0.880}$

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span; $L = \mathbf{5000}$ mm

Length of segment for LT buckling; $L_{LT} = \mathbf{5000}$ mm

Compression flanges laterally restrained

Both flanges free to rotate on plan

Effective length for LT buckling; $L_{E,LT} = L_{LT} \times 1.0 = \mathbf{5000}$ mm

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for uls design

Applied shear force; $F_{vy} = \mathbf{67.1}$ kN

Maximum bending moment; $M_{LT} = M_x = \mathbf{83.90}$ kNm

Applied torque; $T_q = \mathbf{9.96}$ kNm

Minor axis bending moment; $M_y = \mathbf{0}$ kNm

Compression force; $F_c = \mathbf{0}$ kN

Equivalent uniform moment factors

EUM factor (Cl. 4.3.6.6 and T18); $m_{LT} = \mathbf{1.000}$

Torsional deflection analysis

Beam is torsion fixed at each end. (as defined in SCI-P-057 section 2.1.6)

Maximum torque (at supports); $T_o = T_q / 2 = \mathbf{4.98}$ kNm

Average torque between support & centreline; $T_{av} = T_o / 2 = \mathbf{2.49}$ kNm

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Max. angle of twist (at midspan);

$$\phi = T_{av} / (G \times J) \times L / 2 \times 1 \text{ rads} = \mathbf{0.002} \text{ rads}$$

Section classification

$$b_x / t = \mathbf{7.0}$$

$$d_x / t = \mathbf{27.0}$$

$$b_y / t = \mathbf{27.0}$$

$$d_y / t = \mathbf{7.0}$$

$$r_{1sx} = \min(1.0, \max(-1.0, F_c / (2 \times d_x \times t \times p_{yw}))) = \mathbf{0.000}$$

$$r_{1sy} = \min(1.0, \max(-1.0, F_c / (2 \times d_y \times t \times p_{yw}))) = \mathbf{0.000}$$

$$r_{2s} = F_c / (A_g \times p_{yw}) = \mathbf{0.000}$$

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force;

$$F_{vy} = \mathbf{67.1} \text{ kN}$$

Design shear resistance (Cl. 4.2.3);

$$P_{vy} = 0.6 \times p_y \times A_{vy} = \mathbf{1197.0} \text{ kN}$$

Pass - Shear

Moment capacity (x-axis)

Design bending moment;

$$M_x = \mathbf{83.9} \text{ kNm}$$

Moment capacity;

$$M_{cxu} = p_y \times S_x = \mathbf{236.3} \text{ kNm}$$

Moment capacity low shear (Cl. 4.2.5.1);

$$M_{cx} = \min(p_y \times S_x, 1.2 \times p_y \times Z_x) = \mathbf{216.2} \text{ kNm}$$

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for lateral torsional buckling;

$$L_{E_LT} = \mathbf{5000} \text{ mm}$$

Slenderness ratio - cl 4.3.6.5;

$$\lambda = L_{E_LT} / r_y = \mathbf{121}$$

$$D / B = \mathbf{3.0}$$

LTB check not required

Buckling resistance moment;

$$M_b = M_{cx} = \mathbf{216.2} \text{ kNm}$$

Buckling under combined bending & torsion - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Maximum angle of twist;

$$\phi = \mathbf{0.002} \text{ rads}$$

Induced minor axis moment;

$$M_{yt} = M_x \times \phi / 1 \text{ rad} = \mathbf{0.18} \text{ kNm}$$

Normal stress at corner due to M_{yt} ;

$$\sigma_{byt} = M_{yt} / Z_y = \mathbf{1} \text{ N/mm}^2$$

Interaction index;

$$i_b = M_x \times m_{LT} / M_b + \sigma_{byt} / p_y \times (1 + 0.5 \times M_x \times m_{LT} / M_b) = \mathbf{0.39}$$

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x ;

$$\sigma_{bx} = M_x / Z_x = \mathbf{165} \text{ N/mm}^2$$

Combined stress - eqn 2.22;

$$\sigma_{bx} + \sigma_{byt} = \mathbf{166} \text{ N/mm}^2$$

Design strength;

$$p_y = \mathbf{355} \text{ N/mm}^2$$

Pass - Local capacity

Combined shear stresses SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. shear stress due to bending;

$$\tau_{bw} = F_{vy} \times Q_w / (I_x \times 2 \times t) = \mathbf{15} \text{ N/mm}^2$$

Max. shear stresses due to torsion;

$$\tau_t = T_o / C = \mathbf{11} \text{ N/mm}^2$$

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Amplified shear stress due to torsion;

$$\tau_{vt} = \tau_t \times (1 + 0.5 \times M_x \times m_{LT} / M_b) = \mathbf{13 \text{ N/mm}^2}$$

Combined shear due to bending & torsion;

$$\tau = \tau_{bw} + \tau_{vt} = \mathbf{28 \text{ N/mm}^2}$$

Shear strength;

$$p_v = 0.6 \times p_y = \mathbf{213 \text{ N/mm}^2}$$

Pass - Combined shear stresses

Twist check

Total applied torque (unfactored);

$$T_{qu} = \mathbf{7.12 \text{ kNm}}$$

Maximum twist under sls loading;

$$\phi_{sls} = \phi \times T_{qu} / T_q = \mathbf{0.09 \text{ deg}}$$

Twist limit;

$$\phi_{lim} = \mathbf{2.00 \text{ deg}}$$

Pass - Twist

Deflection

Maximum y-axis deflection;

$$\delta_{y_max} = \mathbf{9.7 \text{ mm}}$$

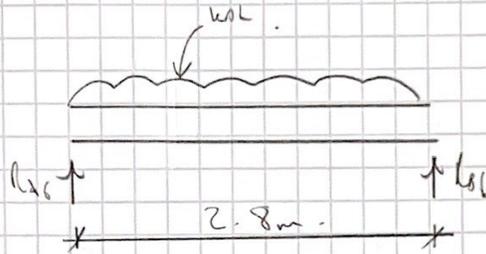
Deflection limit - cl. 2.5.2;

$$\delta_{lim} = \min(L / k_\delta, \delta_{lim_abs}) = \mathbf{10.0 \text{ mm}}$$

Pass - Deflection within specified limit

;

— MKG - BEAM + PLATE + CURVE SPAN 2.8m S355 STEEL.



WAL LOADS - INNER.

- gk • 100mm BLOCK + PLYSTYROFOAM (1.95kN/m²) x HEIGHT 1.45m
- ROOF (0.6kN/m²) x COVER WIDTH 3.6m / 2
- qk • ROOF (0.6kN/m²) x COVER WIDTH 3.6m / 2.

SEW -

2.85

1.8

1.8

WAL OVER.

- gk • 100mm BLOCK + CONCRETE (2.1kN/m²) x HEIGHT 1.45m.

3.05

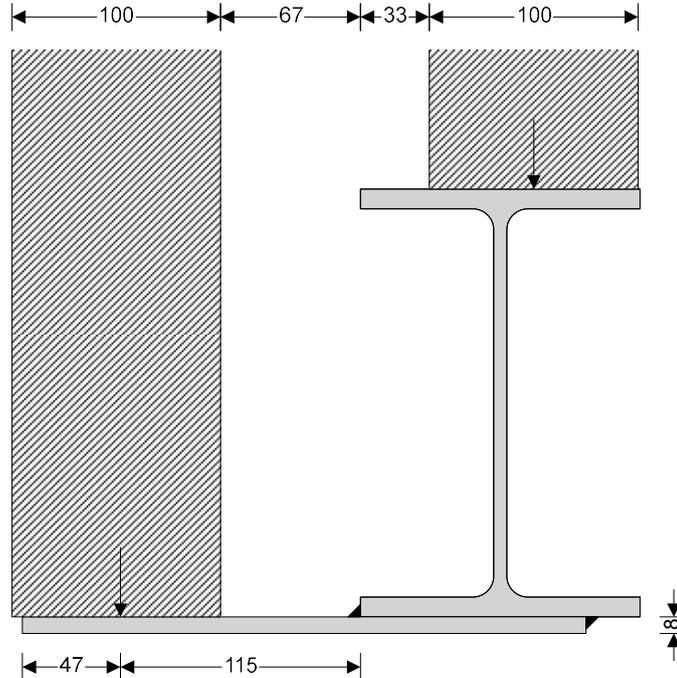
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MK6 STEEL MASONRY SUPPORT (BS5950)

STEEL MASONRY SUPPORT

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

Tedds calculation version 1.0.04



Steel member details

Torsion beam;	UKB 203x133x30
Masonry support plate;	User
Steel grade of support plate;	S355
Design strength of support plate;	$p_{ysb} = 355 \text{ N/mm}^2$
Modulus of elasticity;	$E = 205000 \text{ N/mm}^2$
Constant;	$\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_{ysb})} = 0.880$
Length of plate beyond beam;	$l_h = 162 \text{ mm}$
Total length of plate;	$l_{plate} = 270 \text{ mm}$
Thickness of plate;	$t_{sb} = 8 \text{ mm}$
Width of main beam;	$B_{mb} = 134 \text{ mm}$
Area of plate;	$A_{sbu} = t_{sb} \times l_{plate} = 2160.0 \text{ mm}^2$
Distance from weld position to CoG;	$c_{yysb} = l_h / 2 - (l_{plate} - l_h) / 2 = 27 \text{ mm}$

Supported materials detail

Density of masonry on main beam;	$\rho_{m,mb} = 20.0 \text{ kN/m}^3$
Width of masonry on main beam;	$b_{mmb} = 100 \text{ mm}$
Height of masonry on main beam;	$h_{mmb} = 1450 \text{ mm}$
Eccentricity of main beam material;	$e_{mb} = 33 \text{ mm}$
Add dead force main beam (not from masonry);	$P_{Gaddmb} = 1.4 \text{ kN/m}$
Add live force main beam (not from masonry);	$P_{Qaddmb} = 1.4 \text{ kN/m}$
Density of masonry on support beam;	$\rho_{m,sb} = 22.0 \text{ kN/m}^3$
Width of masonry on support beam;	$b_{msb} = 100 \text{ mm}$

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Height of masonry on support beam; $h_{msb} = 1450$ mm
 Add dead force support beam (not from masonry); $P_{Gaddsb} = 0.0$ kN/m
 Add live force support beam (not from masonry); $P_{Qaddsb} = 0.0$ kN/m

Geometry

Cavity width; $c = 100$ mm
 Supported width of masonry; $d_m = l_h + e_{mb} - c = 95$ mm

Biaxial stress effects in the plate (SCI-P-110)

Maximum overall bending moment; $M_x = 13.0$ kNm
 Dist to NA combined section (CoG torsion beam); $y_{e,all} = (D_{mb} + t_{sb}) \times A_{sbu} / (2 \times (A_{mb} + A_{sbu})) = 39$ mm
 Second moment of area of combined section; $I_{xx,all} = (I_{xx,mb} + A_{mb} \times y_{e,all}^2) + A_{sbu} \times (D_{mb} / 2 + t_{sb} / 2 - y_{e,all})^2 = 4487$ cm⁴
 Elastic section modulus of combined section; $Z_{xx,all} = I_{xx,all} / (D_{mb} / 2 + t_{sb} - y_{e,all}) = 617.97$ cm³
 Section modulus of plate; $Z_{xx,plate} = 1m \times t_{sb}^2 / (6 \times 1m) = 10.67$ cm³/m
 Eccentricity of support beam masonry; $e_1 = 115$ mm
 Force of masonry on support plate; $P_1 = (b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ} = 4.5$ kN/m
 Bending at heel; $M_{x,plate} = P_1 \times e_1 = 0.5$ kNm/m
 Moment capacity of plate; $M_c = 1.2 \times Z_{x,plate} \times p_{ysb} = 4.5$ kNm/m

PASS - Design strength exceeds stress at heel

Longitudinal stress due to overall bending; $\sigma_1 = M_x / Z_{xx,all} = 21.0$ N/mm²
 Constant relating to Von Mises curve; $C_{fp} = (4 \times p_{ysb}^2 - 3 \times \sigma_1^2)^{0.5} = 709.1$ N/mm²
 Transverse bending stress ratio limit; $\alpha_{ts} = (C_{fp}^2 - \sigma_1^2) / (2 \times C_{fp} \times p_{ysb}) = 0.998$
 Transverse bending stress ratio; $\alpha_{ts} = M_{x,plate} / M_c = 0.113$

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfactored force on support angle; $P_{1SLS} = b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb} = 3.2$ kN/m
 Distance from weld to load position; $a_m = e_1 = 115$ mm
 Length of load resultant to edge of plate; $b_m = l_h - e_1 = 47$ mm
 Dist from weld to load position as ratio of length; $a_l = a_m / (a_m + b_m) = 0.710$
 Effective second moment of inertia; $I_{eff_def} = t_{sb}^3 / 12 = 42667$ mm⁴/m
 Deflection at toe; $\delta = (a_l^2 \times (3 - a_l) / 6) \times (P_{1SLS} \times (a_m + b_m)^3) / (E_{S5950} \times I_{eff_def}) = 0.30$ mm
 Deflection limit; $\delta_{lim} = 1.95$ mm

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Leg length of weld; $s_{weld} = 6$ mm
 Throat size of weld; $a_{weld} = 1/\sqrt{2} \times s_{weld} = 4.2$ mm
 Shear force at weld position; $R_A = P_1 \times \max((1 + (3 \times e_1) / (2 \times B_{mb} / 2)), 1.4) = 16.0$ kN/m
 Maximum possible force in plate; $R_p = (l_h + B_{mb}) \times t_{sb} \times p_{ysb} = 840.4$ kN
 Longitudinal shear between beam and plate; $R_l = 2 \times R_p / L = 600.3$ kN/m
 Horizontal shear between beam and plate; $R_h = P_1 \times e_1 / (s_{weld} / 2 + t_{sb} / 2) = 73.4$ kN/m
 Resultant weld force; $R_{weld} = (R_A^2 + R_l^2 + R_h^2)^{0.5} = 0.605$ kN/mm
 Strength of weld (Table 37); $p_{weld} = 220.0$ N/mm²
 Capacity of full length weld; $p_{c,weld} = a_{weld} \times p_{weld} = 0.933$ kN/mm

PASS - Capacity of weld exceeds resultant force on weld

Torsional loading ULS

Loading of support beam masonry; $w_{1ULS} = (h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ} = 4.47$ kN/m

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Loading of main beam masonry;

$$W_{2ULS} = (h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb}) \times \gamma_{fG} + P_{Qaddmb} \times \gamma_{fQ} = \mathbf{8.11 \text{ kN/m}}$$

Self weight of support beam;

$$W_{3ULS} = A_{sbu} \times \rho_{sb} \times \gamma_{fG} = \mathbf{0.24 \text{ kN/m}}$$

Torsional loading SLS

Loading of support beam masonry;

$$W_{1SLS} = h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb} = \mathbf{3.19 \text{ kN/m}}$$

Loading of main beam masonry;

$$W_{2SLS} = h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb} + P_{Qaddmb} = \mathbf{5.60 \text{ kN/m}}$$

Self weight of support beam;

$$W_{3SLS} = A_{sbu} \times \rho_{sb} = \mathbf{0.17 \text{ kN/m}}$$

Eccentricities

Distance to shear centre of main beam;

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

Eccentricity of support beam masonry;

$$e_{1mb} = (B_{mb} + b_{msb}) / 2 + c - e_{mb} = \mathbf{184 \text{ mm}}$$

Eccentricity of main beam masonry;

$$e_{2mb} = (B_{mb} - b_{mmb}) / 2 - e_{mb} = \mathbf{-16 \text{ mm}}$$

Eccentricity of support beam;

$$e_{3mb} = B_{mb} / 2 + c_{yysb} = \mathbf{94 \text{ mm}}$$

Torsional effects

Applied torque (ULS);

$$T_{qULS} = \text{abs}(W_{1ULS} \times e_{1mb} + W_{2ULS} \times e_{2mb} + W_{3ULS} \times e_{3mb}) = \mathbf{0.71 \text{ kNm/m}}$$

Total torque (ULS);

$$T_q = T_{qULS} \times L = \mathbf{2.00 \text{ kNm}}$$

Applied torque (SLS);

$$T_{qSLS} = \text{abs}(W_{1SLS} \times e_{1mb} + W_{2SLS} \times e_{2mb} + W_{3SLS} \times e_{3mb}) = \mathbf{0.51 \text{ kNm/m}}$$

Total torque (SLS);

$$T_{qu} = T_{qSLS} \times L = \mathbf{1.44 \text{ kNm}}$$

STEEL BEAM TORSION DESIGN

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

Tedds calculation version 2.0.02

Section details

Section type;

UKB 203x133x30

Steel grade;

S355

Design strength;

$$p_{yw} = p_y = \mathbf{355 \text{ N/mm}^2}$$

Constant;

$$\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_y)} = \mathbf{0.880}$$

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span;

$$L = \mathbf{2800 \text{ mm}}$$

Length of segment for LT buckling;

$$L_{LT} = \mathbf{2800 \text{ mm}}$$

Compression flanges laterally restrained

Both flanges free to rotate on plan

Effective length for LT buckling;

$$L_{E,LT} = L_{LT} \times 1.0 = \mathbf{2800 \text{ mm}}$$

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for uls design

Applied shear force;

$$F_{vy} = \mathbf{18.5 \text{ kN}}$$

Maximum bending moment;

$$M_{LT} = M_x = \mathbf{12.97 \text{ kNm}}$$

Applied torque;

$$T_q = \mathbf{2.00 \text{ kNm}}$$

Minor axis bending moment;

$$M_y = \mathbf{0 \text{ kNm}}$$

Compression force;

$$F_c = \mathbf{0 \text{ kN}}$$

Equivalent uniform moment factors

EUM factor (Cl. 4.3.6.6 and T18);

$$m_{LT} = \mathbf{1.000}$$

Torsional deflection parameters

Beam is torsion fixed and warping free at each end. (as defined in SCI-P-057 section 2.1.6) - Appendix B case 4

Dist along the beam for first derivative of twist;

$$z_1 = \mathbf{0 \text{ mm}}$$

Dist along the beam for second derivative of twist;

$$z_2 = L / 2 = \mathbf{1400 \text{ mm}}$$

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First derivative of angle of twist;	$\phi'_1 = T_q / (G \times J) \times a / L \times [L^2 / (2 \times a) \times (1 / L - 2 \times z_1 / L^2) + \sinh(z_1 / a) - \tanh(L / (2 \times a)) \times \cosh(z_1 / a)] \times 1 \text{ rads} = \mathbf{4.67 \times 10^{-2} \text{ rads/m}}$
Third derivative of angle of twist;	$\phi'''_1 = T_q / (G \times J \times a^2) \times a / L \times [\sinh(z_1 / a) - \tanh(L / (2 \times a)) \times \cosh(z_1 / a)] \times 1 \text{ rads} = \mathbf{-8.09 \times 10^{-2} \text{ rads/m}^3}$
Angle of twist;	$\phi_2 = T_q \times a / (G \times J) \times a / L \times [L^2 / (2 \times a^2) \times (z_2 / L - z_2^2 / L^2) + \cosh(z_2 / a) - \tanh(L / (2 \times a)) \times \sinh(z_2 / a) - 1] \times 1 \text{ rads} = \mathbf{0.040 \text{ rads}}$
Second derivative of angle of twist;	$\phi''_2 = T_q / (G \times J \times a) \times a / L \times [\cosh(z_2 / a) - \tanh(L / (2 \times a)) \times \sinh(z_2 / a) - 1] \times 1 \text{ rads} = \mathbf{-4.85 \times 10^{-2} \text{ rads/m}^2}$

Design parameters

Total angle of twist;	$\phi = \text{abs}(\phi_2) = \mathbf{0.040 \text{ rads}}$
First derivative of ϕ ;	$\phi' = \text{abs}(\phi'_1) = \mathbf{4.67 \times 10^{-2} \text{ rads/m}}$
Second derivative of ϕ ;	$\phi'' = \text{abs}(\phi''_2) = \mathbf{4.85 \times 10^{-2} \text{ rads/m}^2}$
Third derivative of ϕ ;	$\phi''' = \text{abs}(\phi'''_1) = \mathbf{8.09 \times 10^{-2} \text{ rads/m}^3}$

Section classification

$$b / T = \mathbf{7.0}$$

$$d / t = \mathbf{26.9}$$

$$r_{1s} = \min(1.0, \max(-1.0, F_c / (d \times t \times p_{yw}))) = \mathbf{0.000}$$

$$r_{2s} = F_c / (A_g \times p_{yw}) = \mathbf{0.000}$$

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force;	$F_{vy} = \mathbf{18.5 \text{ kN}}$
Design shear resistance (Cl. 4.2.3);	$P_{vy} = 0.6 \times p_y \times A_{vy} = \mathbf{281.9 \text{ kN}}$

Pass - Shear

Moment capacity (x-axis)

Design bending moment;	$M_x = \mathbf{13.0 \text{ kNm}}$
Moment capacity;	$M_{cxu} = p_y \times S_x = \mathbf{111.6 \text{ kNm}}$
Moment capacity low shear (Cl. 4.2.5.1);	$M_{cx} = \min(p_y \times S_x, 1.2 \times p_y \times Z_x) = \mathbf{111.6 \text{ kNm}}$

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for lateral torsional buckling;	$L_{E_LT} = \mathbf{2800 \text{ mm}}$
Slenderness ratio;	$\lambda = L_{E_LT} / r_y = \mathbf{88}$
Buckling parameter;	$u = \mathbf{0.881}$
Flange ratio;	$\eta = \mathbf{0.5}$
Torsional index;	$x = \mathbf{21.5}$
Slenderness factor;	$v = 1 / (1 + 0.05 \times (\lambda / x)^2)^{0.25} = \mathbf{0.86}$
Ratio - cl 4.3.6.9;	$\beta_w = 1.0 = \mathbf{1.000}$
Equivalent slenderness – cl 4.3.6.7;	$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = \mathbf{67}$
Limiting slenderness – Annex B2.2;	$\lambda_{L0} = 0.4 \times \sqrt{(\pi^2 \times E_{S5950} / p_y)} = \mathbf{30}$
Euler stress;	$p_E = \pi^2 \times E_{S5950} / \lambda_{LT}^2 = \mathbf{454 \text{ N/mm}^2}$
Perry factor;	$\eta_{LT} = \max(7.0 \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.256}$
Bending strength;	$p_b = p_E \times p_y / (\phi_{LT} + \sqrt{(\phi_{LT}^2 - p_E \times p_y)}) = \mathbf{233 \text{ N/mm}^2}$
Buckling resistance moment;	$M_b = p_b \times S_x = \mathbf{73.2 \text{ kNm}}$
Max moment governing buckling resistance;	$M_{LT} = \mathbf{13.0 \text{ kNm}}$
Equiv uniform moment factor for LTB;	$m_{LT} = \mathbf{1.00}$

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$$M_b / m_{LT} = 73.2 \text{ kNm}$$

Pass - lat. tors. buckling

Buckling under combined bending & torsion -SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Span factor;	$L / a = 2.88$
Angle of twist;	$\phi = 0.040 \text{ rads}$
Second derivative of ϕ ;	$\phi'' = 48.5 \times 10^{-3} \text{ rads/m}^2$
Induced minor axis moment;	$M_{yt} = M_x \times \phi / 1 \text{ rad} = 0.52 \text{ kNm}$
Normal stress at flange tip due to M_{yt} ;	$\sigma_{byt} = M_{yt} / Z_y = 9 \text{ N/mm}^2$
Normal stress at flange tip due to warping;	$\sigma_w = E_{S5950} \times W_{n0} \times \phi'' / 1 \text{ rad} = 66 \text{ N/mm}^2$
Interaction index;	$i_b = M_x \times m_{LT} / M_b + (\sigma_{byt} + \sigma_w) / p_y \times (1 + 0.5 \times M_x \times m_{LT} / M_b) = 0.41$

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x ;	$\sigma_{bx} = M_x / Z_x = 46 \text{ N/mm}^2$
Combined stress - eqn 2.22;	$\sigma_{bx} + \sigma_{byt} + \sigma_w = 121 \text{ N/mm}^2$
Design strength;	$p_y = 355 \text{ N/mm}^2$

Pass - Local capacity

Combined shear stresses - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max shear stresses due to bending in web;	$\tau_{bw} = F_{vy} \times Q_w / (I_x \times t) = 16 \text{ N/mm}^2$
Max shear stresses due to bending in flange;	$\tau_{bf} = F_{vy} \times Q_f / (I_x \times T) = 4 \text{ N/mm}^2$
Max shear stresses due to torsion in web;	$\tau_{tw} = \text{abs}(G \times t \times \phi' / 1 \text{ rad}) = 24 \text{ N/mm}^2$
Max shear stresses due to torsion in flange;	$\tau_{tf} = \text{abs}(G \times T \times \phi' / 1 \text{ rad}) = 35 \text{ N/mm}^2$
Max shear stresses due to warping in flange;	$\tau_{wf} = \text{abs}(-E_{S5950} \times S_{w1} \times \phi''' / 1 \text{ rad} / T) = 4 \text{ N/mm}^2$
Amp shear stress torsion & warping in web;	$\tau_{vtw} = \tau_{tw} \times (1 + 0.5 \times M_x \times m_{LT} / M_b) = 26 \text{ N/mm}^2$
Amp shear stress torsion & warping in flange;	$\tau_{vtf} = (\tau_{tf} + \tau_{wf}) \times (1 + 0.5 \times M_x \times m_{LT} / M_b) = 42 \text{ N/mm}^2$

Combined shear stresses due to bending, torsion & warping:

Combined shear stresses in web;	$\tau_w = \tau_{bw} + \tau_{vtw} = 41 \text{ N/mm}^2$
Combined shear stresses in flange;	$\tau_f = \tau_{bf} + \tau_{vtf} = 47 \text{ N/mm}^2$
Shear strength;	$p_v = 0.6 \times p_y = 213 \text{ N/mm}^2$

Pass - Combined shear stresses

Twist check

Total applied torque (unfactored);	$T_{qu} = 1.44 \text{ kNm}$
Maximum twist under sls loading;	$\phi_{sls} = \phi \times T_{qu} / T_q = 1.66 \text{ deg}$
Twist limit;	$\phi_{lim} = 2.00 \text{ deg}$

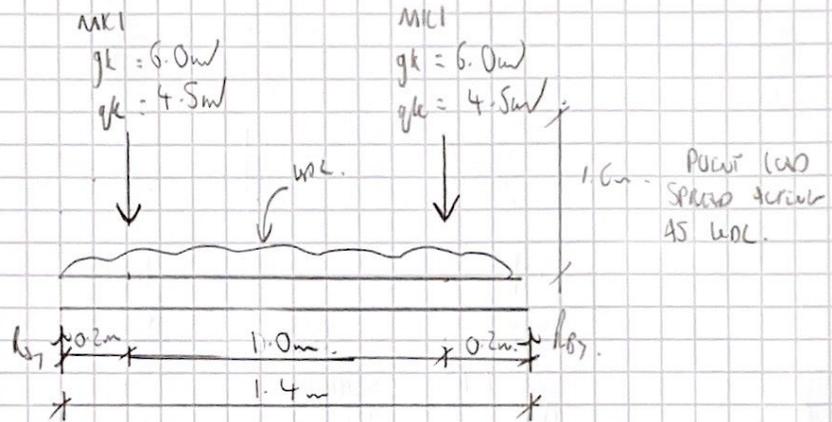
Pass - Twist

Deflection

Maximum y-axis deflection;	$\delta_{y_max} = 1.2 \text{ mm}$
Deflection limit - cl. 2.5.2;	$\delta_{lim} = \min(L / k_\delta, \delta_{lim_abs}) = 5.0 \text{ mm}$

Pass - Deflection within specified limit

Retn MK7 - UTRC SPAN 1.4m S355 STEEL.



UCL COVER - UCL

SEW.

q_k : 100mm block + plasterboard. $(1.95 \text{ kN/m}^2) \times \text{COVER HEIGHT } 1.6\text{m}$
 $= 12.0 \text{ kN} / 1.6\text{m}$
 q_k : 9.0 / 1.6m.
UCL COVER OVER.

3.15 kN/m
 7.5 kN/m
 5.6 kN/m

q_k : 100mm block + plasterboard $(2.1 \text{ kN/m}^2) \times \text{COVER HEIGHT } 1.6\text{m}$

3.36 kN/m

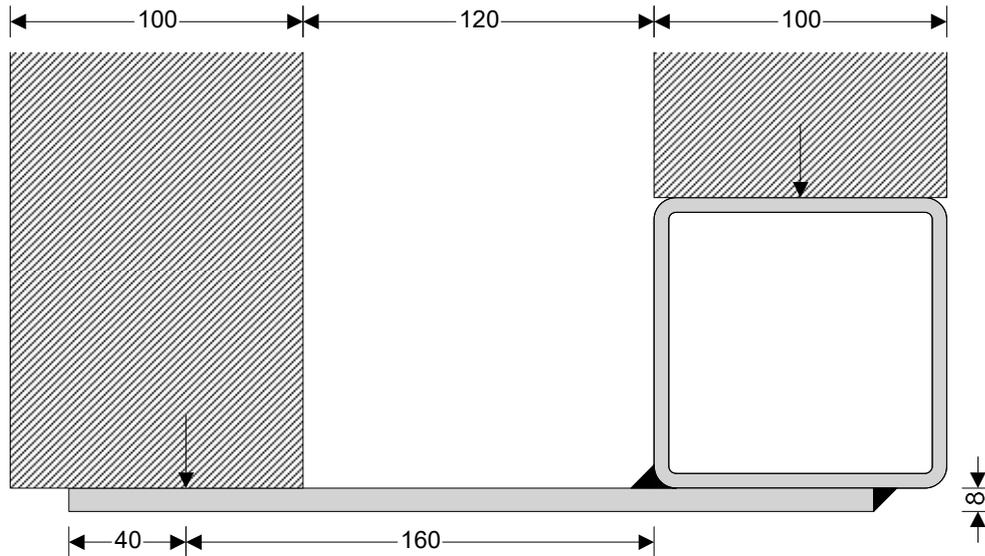
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MK7 STEEL MASONRY SUPPORT (BS5950)

STEEL MASONRY SUPPORT

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

Tedds calculation version 1.0.04



Steel member details

Torsion beam;	SHS 100x100x5.0
Masonry support plate;	User
Steel grade of support plate;	S355
Design strength of support plate;	$p_{ysb} = 355 \text{ N/mm}^2$
Modulus of elasticity;	$E = 205000 \text{ N/mm}^2$
Constant;	$\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_{ysb})} = 0.880$
Length of plate beyond beam;	$l_h = 200 \text{ mm}$
Total length of plate;	$l_{plate} = 275 \text{ mm}$
Thickness of plate;	$t_{sb} = 8 \text{ mm}$
Width of main beam;	$B_{mb} = 100 \text{ mm}$
Area of plate;	$A_{sbu} = t_{sb} \times l_{plate} = 2200.0 \text{ mm}^2$
Distance from weld position to CoG;	$c_{yysb} = l_h / 2 - (l_{plate} - l_h) / 2 = 63 \text{ mm}$

Supported materials detail

Density of masonry on main beam;	$\rho_{m,mb} = 20.0 \text{ kN/m}^3$
Width of masonry on main beam;	$b_{mmb} = 100 \text{ mm}$
Height of masonry on main beam;	$h_{mmb} = 1600 \text{ mm}$
Eccentricity of main beam material;	$e_{mb} = 0 \text{ mm}$
Add dead force main beam (not from masonry);	$P_{Gaddmb} = 7.5 \text{ kN/m}$
Add live force main beam (not from masonry);	$P_{Qaddmb} = 5.6 \text{ kN/m}$
Density of masonry on support beam;	$\rho_{m,sb} = 22.0 \text{ kN/m}^3$
Width of masonry on support beam;	$b_{msb} = 100 \text{ mm}$
Height of masonry on support beam;	$h_{msb} = 1600 \text{ mm}$
Add dead force support beam (not from masonry);	$P_{Gaddsb} = 0.0 \text{ kN/m}$

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Add live force support beam (not from masonry); $P_{Qaddsb} = 0.0$ kN/m

Geometry

Cavity width; $c = 120$ mm
Supported width of masonry; $d_m = l_h + e_{mb} - c = 80$ mm

Biaxial stress effects in the plate (SCI-P-110)

Maximum overall bending moment; $M_x = 10.3$ kNm
Dist to NA combined section (CoG torsion beam); $y_{e,all} = (D_{mb} + t_{sb}) \times A_{sbu} / (2 \times (A_{mb} + A_{sbu})) = 29$ mm
Second moment of area of combined section; $I_{xx,all} = (I_{xxmb} + A_{mb} \times y_{e,all}^2) + A_{sbu} \times (D_{mb} / 2 + t_{sb} / 2 - y_{e,all})^2 = 574$ cm⁴
Elastic section modulus of combined section; $Z_{xx,all} = I_{xx,all} / (D_{mb} / 2 + t_{sb} - y_{e,all}) = 199.23$ cm³
Section modulus of plate; $Z_{xx,plate} = 1m \times t_{sb}^2 / (6 \times 1m) = 10.67$ cm³/m
Eccentricity of support beam masonry; $e_1 = 160$ mm
Force of masonry on support plate; $P_1 = (b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ} = 4.9$ kN/m
Bending at heel; $M_{x,plate} = P_1 \times e_1 = 0.8$ kNm/m
Moment capacity of plate; $M_c = 1.2 \times Z_{x,plate} \times p_{ysb} = 4.5$ kNm/m

PASS - Design strength exceeds stress at heel

Longitudinal stress due to overall bending; $\sigma_1 = M_x / Z_{xx,all} = 51.6$ N/mm²
Constant relating to Von Mises curve; $C_{fp} = (4 \times p_{ysb}^2 - 3 \times \sigma_1^2)^{0.5} = 704.4$ N/mm²
Transverse bending stress ratio limit; $\alpha_{ts} = (C_{fp}^2 - \sigma_1^2) / (2 \times C_{fp} \times p_{ysb}) = 0.987$
Transverse bending stress ratio; $\alpha_{ls} = M_{x,plate} / M_c = 0.174$

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfactored force on support angle; $P_{1SLS} = b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb} = 3.5$ kN/m
Distance from weld to load position; $a_m = e_1 = 160$ mm
Length of load resultant to edge of plate; $b_m = l_h - e_1 = 40$ mm
Dist from weld to load position as ratio of length; $a_l = a_m / (a_m + b_m) = 0.800$
Effective second moment of inertia; $I_{eff_def} = t_{sb}^3 / 12 = 42667$ mm⁴/m
Deflection at toe; $\delta = (a_l^2 \times (3 - a_l) / 6) \times (P_{1SLS} \times (a_m + b_m)^3) / (E_{S5950} \times I_{eff_def}) = 0.76$ mm
Deflection limit; $\delta_{lim} = 1.67$ mm

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Leg length of weld; $s_{weld} = 8$ mm
Throat size of weld; $a_{weld} = 1/\sqrt{2} \times s_{weld} = 5.7$ mm
Shear force at weld position; $R_A = P_1 \times \max((1 + (3 \times e_1) / (2 \times B_{mb} / 2)), 1.4) = 28.6$ kN/m
Maximum possible force in plate; $R_p = (l_h + B_{mb}) \times t_{sb} \times p_{ysb} = 852.0$ kN
Longitudinal shear between beam and plate; $R_l = 2 \times R_p / L = 1217.1$ kN/m
Horizontal shear between beam and plate; $R_h = P_1 \times e_1 / (s_{weld} / 2 + t_{sb} / 2) = 98.6$ kN/m
Resultant weld force; $R_{weld} = (R_A^2 + R_l^2 + R_h^2)^{0.5} = 1.221$ kN/mm
Strength of weld (Table 37); $p_{weld} = 220.0$ N/mm²
Capacity of full length weld; $p_{c,weld} = a_{weld} \times p_{weld} = 1.245$ kN/mm

PASS - Capacity of weld exceeds resultant force on weld

Torsional loading ULS

Loading of support beam masonry; $W_{1ULS} = (h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ} = 4.93$ kN/m
Loading of main beam masonry; $W_{2ULS} = (h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb}) \times \gamma_{fG} + P_{Qaddmb} \times \gamma_{fQ} = 23.94$ kN/m

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Self weight of support beam;

$$W_{3ULS} = A_{sbu} \times \rho_{sb} \times \gamma_{fG} = \mathbf{0.24 \text{ kN/m}}$$

Torsional loading SLS

Loading of support beam masonry;

$$W_{1SLS} = h_{msb} \times b_{msb} \times \rho_{m, sb} + P_{Gaddsb} + P_{Qaddsb} = \mathbf{3.52 \text{ kN/m}}$$

Loading of main beam masonry;

$$W_{2SLS} = h_{mmb} \times b_{mmb} \times \rho_{m, mb} + P_{Gaddmb} + P_{Qaddmb} = \mathbf{16.30 \text{ kN/m}}$$

Self weight of support beam;

$$W_{3SLS} = A_{sbu} \times \rho_{sb} = \mathbf{0.17 \text{ kN/m}}$$

Eccentricities

Distance to shear centre of main beam;

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

Eccentricity of support beam masonry;

$$e_{1mb} = (B_{mb} + b_{msb}) / 2 + c - e_{mb} = \mathbf{220 \text{ mm}}$$

Eccentricity of main beam masonry;

$$e_{2mb} = (B_{mb} - b_{mmb}) / 2 - e_{mb} = \mathbf{0 \text{ mm}}$$

Eccentricity of support beam;

$$e_{3mb} = B_{mb} / 2 + C_{yy sb} = \mathbf{113 \text{ mm}}$$

Torsional effects

Applied torque (ULS);

$$T_{qULS} = \text{abs}(W_{1ULS} \times e_{1mb} + W_{2ULS} \times e_{2mb} + W_{3ULS} \times e_{3mb}) = \mathbf{1.11 \text{ kNm/m}}$$

Total torque (ULS);

$$T_q = T_{qULS} \times L = \mathbf{1.56 \text{ kNm}}$$

Applied torque (SLS);

$$T_{qSLS} = \text{abs}(W_{1SLS} \times e_{1mb} + W_{2SLS} \times e_{2mb} + W_{3SLS} \times e_{3mb}) = \mathbf{0.79 \text{ kNm/m}}$$

Total torque (SLS);

$$T_{qu} = T_{qSLS} \times L = \mathbf{1.11 \text{ kNm}}$$

STEEL BEAM TORSION DESIGN

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

Tedds calculation version 2.0.02

Section details

Section type;

SHS 100x100x5.0

Steel grade;

S355

Design strength;

$$p_{yw} = p_y = \mathbf{355 \text{ N/mm}^2}$$

Constant;

$$\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_y)} = \mathbf{0.880}$$

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span;

$$L = \mathbf{1400 \text{ mm}}$$

Length of segment for LT buckling;

$$L_{LT} = \mathbf{1400 \text{ mm}}$$

Compression flanges laterally restrained

Both flanges free to rotate on plan

Effective length for LT buckling;

$$L_{E_LT} = L_{LT} \times 1.0 = \mathbf{1400 \text{ mm}}$$

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for uls design

Applied shear force;

$$F_{vy} = \mathbf{36.1 \text{ kN}}$$

Maximum bending moment;

$$M_{LT} = M_x = \mathbf{10.28 \text{ kNm}}$$

Applied torque;

$$T_q = \mathbf{1.56 \text{ kNm}}$$

Minor axis bending moment;

$$M_y = \mathbf{0 \text{ kNm}}$$

Compression force;

$$F_c = \mathbf{0 \text{ kN}}$$

Equivalent uniform moment factors

EUM factor (Cl. 4.3.6.6 and T18);

$$m_{LT} = \mathbf{1.000}$$

Torsional deflection analysis

Beam is torsion fixed at each end. (as defined in SCI-P-057 section 2.1.6)

Maximum torque (at supports);

$$T_o = T_q / 2 = \mathbf{0.78 \text{ kNm}}$$

Average torque between support & centreline;

$$T_{av} = T_o / 2 = \mathbf{0.39 \text{ kNm}}$$

Max. angle of twist (at midspan);

$$\phi = T_{av} / (G \times J) \times L / 2 \times 1 \text{ rads} = \mathbf{0.001 \text{ rads}}$$

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Section classification

$$b_x / t = 17.0$$

$$d_x / t = 17.0$$

$$b_y / t = 17.0$$

$$d_y / t = 17.0$$

$$r_{1sx} = \min(1.0, \max(-1.0, F_c / (2 \times d_x \times t \times p_{yw}))) = 0.000$$

$$r_{1sy} = \min(1.0, \max(-1.0, F_c / (2 \times d_y \times t \times p_{yw}))) = 0.000$$

$$r_{2s} = F_c / (A_g \times p_{yw}) = 0.000$$

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force;

$$F_{vy} = 36.1 \text{ kN}$$

Design shear resistance (Cl. 4.2.3);

$$P_{vy} = 0.6 \times p_y \times A_{vy} = 199.5 \text{ kN}$$

Pass - Shear

Moment capacity (x-axis)

Design bending moment;

$$M_x = 10.3 \text{ kNm}$$

Moment capacity;

$$M_{cxu} = p_y \times S_x = 23.6 \text{ kNm}$$

Moment capacity low shear (Cl. 4.2.5.1);

$$M_{cx} = \min(p_y \times S_x, 1.2 \times p_y \times Z_x) = 23.6 \text{ kNm}$$

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

LT buckling check not required for this section (cl. 4.6.3.1)

Buckling resistance moment;

$$M_b = M_{cx} = 23.6 \text{ kNm}$$

LT buckling check not required for this section

Buckling under combined bending & torsion - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Maximum angle of twist;

$$\phi = 0.001 \text{ rads}$$

Induced minor axis moment;

$$M_{yt} = M_x \times \phi / 1 \text{ rad} = 0.01 \text{ kNm}$$

Normal stress at corner due to M_{yt} ;

$$\sigma_{byt} = M_{yt} / Z_y = 0 \text{ N/mm}^2$$

Interaction index;

$$i_b = M_x \times m_{LT} / M_b + \sigma_{byt} / p_y \times (1 + 0.5 \times M_x \times m_{LT} / M_b) = 0.44$$

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x ;

$$\sigma_{bx} = M_x / Z_x = 184 \text{ N/mm}^2$$

Combined stress - eqn 2.22;

$$\sigma_{bx} + \sigma_{byt} = 184 \text{ N/mm}^2$$

Design strength;

$$p_y = 355 \text{ N/mm}^2$$

Pass - Local capacity

Combined shear stresses SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. shear stress due to bending;

$$\tau_{bw} = F_{vy} \times Q_w / (I_x \times 2 \times t) = 43 \text{ N/mm}^2$$

Max. shear stresses due to torsion;

$$\tau_t = T_o / C = 10 \text{ N/mm}^2$$

Amplified shear stress due to torsion;

$$\tau_{vt} = \tau_t \times (1 + 0.5 \times M_x \times m_{LT} / M_b) = 12 \text{ N/mm}^2$$

Combined shear due to bending & torsion;

$$\tau = \tau_{bw} + \tau_{vt} = 54 \text{ N/mm}^2$$

Shear strength;

$$p_v = 0.6 \times p_y = 213 \text{ N/mm}^2$$

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Pass - Combined shear stresses

Twist check

Total applied torque (unfactored);

$$T_{qu} = 1.11 \text{ kNm}$$

Maximum twist under sls loading;

$$\phi_{sls} = \phi \times T_{qu} / T_q = 0.03 \text{ deg}$$

Twist limit;

$$\phi_{lim} = 2.00 \text{ deg}$$

Pass - Twist

Deflection

Maximum y-axis deflection;

$$\delta_{y_max} = 1.8 \text{ mm}$$

Deflection limit - cl. 2.5.2;

$$\delta_{lim} = \min(L/ k_{\delta}, \delta_{lim_abs}) = 3.9 \text{ mm}$$

Pass - Deflection within specified limit

;

PROPOSED DESIGN

$$\text{Beam MK3} = 29.2 \text{ kN}$$

$$100 \text{ mm block, } 7 \text{ N/mm}^2, F_k = 6.4 \text{ N/mm}^2$$

FIND ALLOWABLE BEARING TO BLOCK.

$$\text{LOS} = 1.25 \times F_k / 3.5$$

$$\text{LOS} = 1.25 \times 6.4 / 3.5$$

$$\text{LOS} = 2.29$$

$$\text{BEARING AREA} = R_s / \text{LOS}$$

$$\frac{29.2 \times 10^3}{2.29} = 12751.1 \text{ mm}^2 / 150 \text{ mm BEARING}$$

$$= 85.0 \text{ mm LONG}$$

∴ PROVIDE 215 mm x 100 mm x 140 mm DEEP C30 CONCRETE PROVISION

$$\text{Beam MK2} = 5.4 \text{ kN}$$

$$\frac{5.4 \times 10^3}{2.29} = 2358 \text{ mm}^2 / 100 \text{ mm BEARING} = 23.6 \text{ mm}$$

∴ PROVIDE 215 mm x 100 mm x 140 mm DEEP C30 CONCRETE PROVISION

BEAM MK7 = 36.1m

$$\frac{36.1m \times 10^3}{2.29} = 15764.2m^2 / 150mm MIN BEAM$$

= 105.1mm \therefore PROVIDE 215mm LOW PROFILE

BEAM MK4 = 57.3m

$$\frac{57.3m \times 10^3}{2.29} = 25021.8m^2 / 100mm MIN BEAM$$

= 250.2mm

\therefore PROVIDE 300mm x 100mm x 140mm DEEP C30 CONCRETE PROFILE

BEAM MK5 = 67.1m

$$\frac{67.1 \times 10^3}{2.29} = 29301.3m^2 / 150mm MIN BEAM$$

= 195.3mm LOW

\therefore PROVIDE 215mm x 100mm x 140mm DEEP C30 CONCRETE PROFILE

BEAM MK6 = 18.5m

$$\frac{18.5m \times 10^3}{2.29} = 8078.6m^2 / 150mm MIN BEAM$$

= 53.9mm LOW

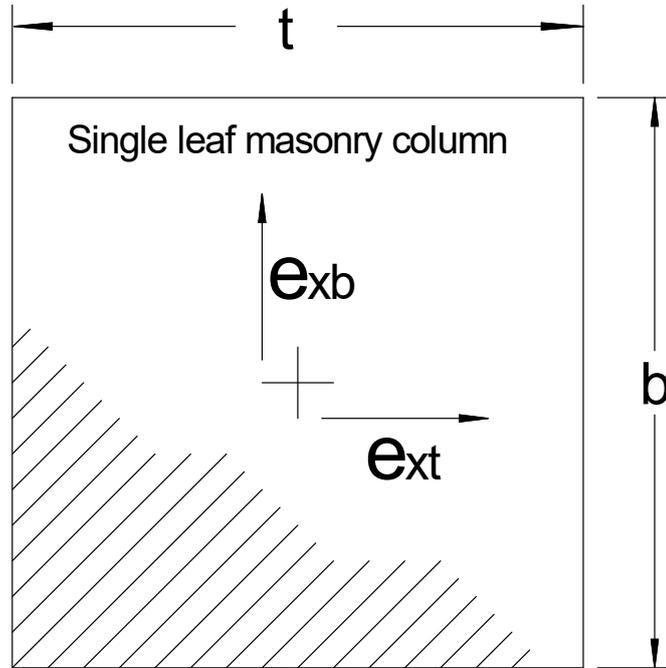
\therefore PROVIDE 215mm x 100mm x 140mm DEEP C30 CONCRETE PROFILE

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MASONRY COLUMN DESIGN (BS5628)

VERTICAL LOADING RECTANGULAR COLUMN (BS5628-1:2005)

TEDDS calculation version 1.0.02



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;	$l_{unit} = 100.0 \text{ mm}$
Block ratio;	$ratio = h_{unit} / l_{unit} = 2.2$

Ratio between 0.6 and 4.5 - OK

Characteristic compressive strength;	$f_k = 6.40 \text{ N/mm}^2$
Column width;	$b = 215 \text{ mm}$
Column thickness;	$t = 800 \text{ mm}$
Column height;	$h = 2.40 \text{ m}$
Column slenderness - minor axis (Clause 24.1);	$\lambda_t = h/t = 3.00$
Column slenderness - major axis (Clause 24.1);	$\lambda_b = h/b = 11.16$
Maximum slenderness;	$\lambda_{max} = \max(\lambda_t, \lambda_b) = 11.16$

Slenderness < 27 - OK

Partial safety factor for material (Table 4);	$\gamma_m = 3.5$
---	------------------

Load eccentricity

Eccentricity of applied load about minor axis;	$e_{xt} = 0.0 \text{ mm}$
Eccentricity of applied load about major axis;	$e_{xb} = 50.0 \text{ mm}$

Capacity reduction factor: minor axis

Eccentricity due to slenderness;	$e_{at} = \max(0 \text{ mm}, t \times (((h/t)^2/2400)-0.015)) = 0.0 \text{ mm}$
Design eccentricity;	$e_{tt} = 0.6 \times \max(\text{abs}(e_{xt}), 0.05 \times t) + e_{at} = 24.0 \text{ mm}$

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Capacity reduction factors;

$$e_{mt} = \max(\text{abs}(e_{xt}), e_{tt}) = \mathbf{24.0 \text{ mm}} ;$$

$$\beta_{tcalc} = \max(0, 1.1 \times (1 - (2 \times e_{mt} / t))) = \mathbf{1.03}$$

$$\beta_{max} = 1.0$$

$$\beta_t = \min(\beta_{tcalc}, \beta_{max}) = \mathbf{1.00}$$

Capacity reduction factor: major axis

Eccentricity due to slenderness;

$$e_{ab} = \max(0 \text{ mm}, b \times (((h/b)^2/2400) - 0.015)) = \mathbf{7.9 \text{ mm}}$$

Design eccentricity;

$$e_{tb} = 0.6 \times \max(\text{abs}(e_{xb}), 0.05 \times b) + e_{ab} = \mathbf{37.9 \text{ mm}}$$

$$e_{mb} = \max(\text{abs}(e_{xb}), e_{tb}) = \mathbf{50.0 \text{ mm}} ;$$

Capacity reduction factors;

$$\beta_{bcalc} = \max(0, 1.1 \times (1 - (2 \times e_{mb} / b))) = \mathbf{0.59}$$

$$\beta_{max} = 1.0$$

$$\beta_b = \min(\beta_{bcalc}, \beta_{max}) = \mathbf{0.59}$$

Minimum capacity reduction factor;

$$\beta_{min} = \min(\beta_t, \beta_b) = \mathbf{0.59}$$

Design vertical load resistance

Compressive strength correction factor

Plan area of column;

$$A = t \times b = \mathbf{0.17 \text{ m}^2}$$

For small plan area (Clause 19.1.2);

$$c = \min(1.0, 0.7 + (1.5 \text{ m}^2) \times A) = \mathbf{0.96}$$

Design vertical load resistance;

$$DVLR = \beta_{min} \times t \times b \times c \times f_k / \gamma_m = \mathbf{177.279 \text{ kN}}$$

Applied factored vertical load on column;

$$V = \mathbf{88.500 \text{ kN}}$$

Column - OK

;
;