

**STRUCTURAL DESIGN CALCULATIONS
FOR
GETHIN HOUSE, MERTHYR TYDFIL
Job Ref: LS378
Date: January 2020**

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**LB STRUCTURAL DESIGN
21 HEOL STRADLING
COITY, BRIDGEND
CF35 6AN**



*The Institution
of Structural
Engineers*



LB Structural Design

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BUILDING LOADS FOR DESIGN CALCULATIONS

Contract No.: **LS378**

Date: Jan 2020

Project: Gethin House

<u>Dead Loads:</u> -		kN/m ²
Roof:	Tiles	0.60
	Felt & battens	0.04
	Rafters	0.12
	Ins felt	0.04
	<u>Plasterboard & skim</u>	<u>0.15</u>
	Total	0.95 kN/m ²
Upper Floors:	Joists & decking	0.40
	<u>Plasterboard & skim</u>	<u>0.20</u>
	Total	0.60 kN/m ²
External walls:	100mm masonry leaf	2.20 kN/m ²

Imposed Loads: -

Roof:	Snow	0.60 kN/m ²
Upper Floors:	Residential	1.50 kN/m ²

Imposed loads are in accordance with BS 6399: Part 1: 1996 - Code of practice for dead and Imposed loads and BS 6399: Part 3: 1988 - Code of practice for imposed roof loads.

SUPER-STRUCTURE CALCULATIONS

TO BE READ IN CONJUNCTION WITH STRUCTURAL REFERENCES AND ARCHITECTS DRAWINGS

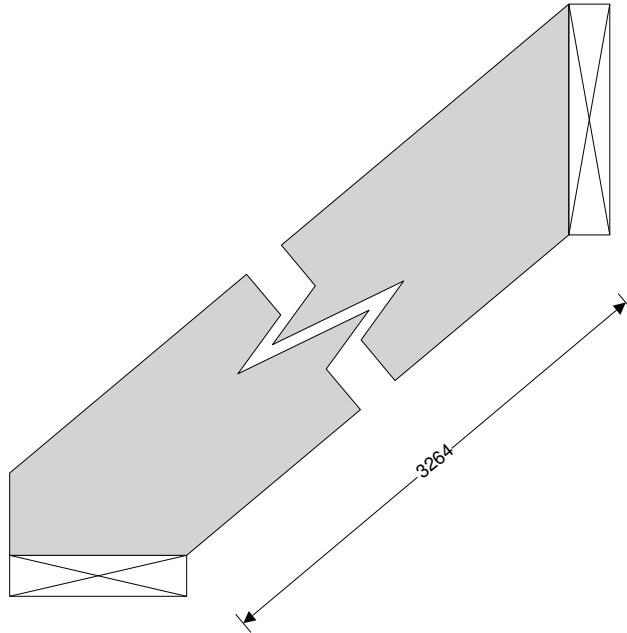
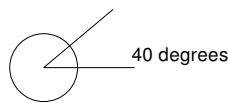
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LB STRUCTURAL DESIGN
21 HEOL STRADLING
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CF35 6AN

Project Gethin House				Job no. LS378	
Calcs for Main Rafters				Start page no./Revision 3	
Calcs by LB	Calcs date 19/12/2019	Checked by LB	Checked date 28/01/2020	Approved by	Approved date

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



Rafter details

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

$b = 45 \text{ mm}$
 $h = 195 \text{ mm}$
 $s = 600 \text{ mm}$
 $\alpha = 40.0 \text{ deg}$
 $L_{clh} = 2500 \text{ mm}$
 $L_{cl} = L_{clh} / \cos(\alpha) = 3264 \text{ mm}$
Single span
C24

Section properties

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

$A = b \times h = 8775 \text{ mm}^2$
 $Z = b \times h^2 / 6 = 285188 \text{ mm}^3$
 $I = b \times h^3 / 12 = 27805781 \text{ mm}^4$
 $r = \sqrt{I / A} = 56.3 \text{ mm}$

Loading details

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

$F_j = b \times h \times \rho_{char} \times g_{acc} = 0.03 \text{ kN/m}$
 $F_d = 0.85 \text{ kN/m}^2$
 $F_u = 0.75 \text{ kN/m}^2$
 $F_p = 0.90 \text{ kN}$

Modification factors

Section depth factor
 Load sharing factor

$K_7 = (300 \text{ mm} / h)^{0.11} = 1.05$
 $K_8 = 1.10$

Consider long term load condition

Load duration factor
 Total UDL perpendicular to rafter

$K_3 = 1.00$
 $F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = 0.414 \text{ kN/m}$

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	LB	19/12/2019	LB	28/01/2020		

Notional bearing length

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

Effective span

$$L_{eff} = L_{cl} + L_b = 3269 \text{ mm}$$

Check bending stress

Bending stress parallel to grain

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

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 8.650 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 1.938 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain

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

Minimum modulus of elasticity

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

Compression member factor

$$K_{12} = 0.68$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 5.910 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.286 \text{ N/mm}^2$$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 21.068 \text{ N/mm}^2$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.986$$

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.276 < 1$$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain

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

Permissible shear stress

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

Applied shear stress

$$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = 0.116 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = 9.808 \text{ mm}$$

Bending deflection

$$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = 2.049 \text{ mm}$$

Shear deflection

$$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = 0.112 \text{ mm}$$

Total deflection

$$\delta_{max} = \delta_b + \delta_s = 2.161 \text{ mm}$$

PASS - Total deflection within permissible limits

Consider medium term load condition

Load duration factor

$$K_3 = 1.25$$

Total UDL perpendicular to rafter

$$F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) = 0.678 \text{ kN/m}$$

Notional bearing length

$$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 9 \text{ mm}$$

Effective span

$$L_{eff} = L_{cl} + L_b = 3273 \text{ mm}$$

Check bending stress

Bending stress parallel to grain

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

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 10.813 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 3.182 \text{ N/mm}^2$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain

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

Minimum modulus of elasticity

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

Compression member factor

$$K_{12} = 0.65$$

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	LB	19/12/2019	LB	28/01/2020		

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{7.068 \text{ N/mm}^2}$$

Applied compressive stress

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = \mathbf{0.469 \text{ N/mm}^2}$$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{21.021 \text{ N/mm}^2}$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = \mathbf{0.978}$$

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = \mathbf{0.367 < 1}$$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain

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

Permissible shear stress

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

Applied shear stress

$$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = \mathbf{0.190 \text{ N/mm}^2}$$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = \mathbf{9.819 \text{ mm}}$$

Bending deflection

$$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = \mathbf{3.372 \text{ mm}}$$

Shear deflection

$$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = \mathbf{0.184 \text{ mm}}$$

Total deflection

$$\delta_{max} = \delta_b + \delta_s = \mathbf{3.556 \text{ mm}}$$

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor

$$K_3 = \mathbf{1.50}$$

Total UDL perpendicular to rafter

$$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = \mathbf{0.414 \text{ kN/m}}$$

Notional bearing length

$$L_b = [F \times L_{cl} + F_p \times \cos(\alpha)] / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = \mathbf{9 \text{ mm}}$$

Effective span

$$L_{eff} = L_{cl} + L_b = \mathbf{3272 \text{ mm}}$$

Check bending stress

Bending stress parallel to grain

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

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{12.976 \text{ N/mm}^2}$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) + F_p \times \cos(\alpha) \times L_{eff} / (4 \times Z) = \mathbf{3.919 \text{ N/mm}^2}$$

PASS - Applied bending stress within permissible limits

Check compressive stress parallel to grain

Compression stress parallel to grain

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

Minimum modulus of elasticity

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

Compression member factor

$$K_{12} = \mathbf{0.62}$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = \mathbf{8.090 \text{ N/mm}^2}$$

Applied compressive stress

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) + F_p \times \sin(\alpha) / A = \mathbf{0.352 \text{ N/mm}^2}$$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = \mathbf{21.031 \text{ N/mm}^2}$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = \mathbf{0.984}$$

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = \mathbf{0.350 < 1}$$

PASS - Combined compressive and bending stresses are within permissible limits

Check shear stress

Shear stress parallel to grain

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

Permissible shear stress

$$\tau_{adm} = \tau \times K_3 \times K_8 = \mathbf{1.172 \text{ N/mm}^2}$$

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	LB	19/12/2019	LB	28/01/2020		

Applied shear stress

$$\tau_{\max} = 3 \times F \times L_{\text{eff}} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A) = \mathbf{0.234 \text{ N/mm}^2}$$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{\text{adm}} = 0.003 \times L_{\text{eff}} = \mathbf{9.816 \text{ mm}}$$

Bending deflection

$$\delta_b = L_{\text{eff}}^3 \times (5 \times F \times L_{\text{eff}} / 384 + F_p \times \cos(\alpha) / 48) / (E_{\text{mean}} \times I) = \mathbf{3.732 \text{ mm}}$$

Shear deflection

$$\delta_s = 12 \times L_{\text{eff}} \times (F \times L_{\text{eff}} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{\text{mean}} \times A) = \mathbf{0.226 \text{ mm}}$$

Total deflection

$$\delta_{\max} = \delta_b + \delta_s = \mathbf{3.959 \text{ mm}}$$

PASS - Total deflection within permissible limits

Project Gethin House				Job no. LS378	
Calcs for Flat Roof Joists				Start page no./Revision 7	
Calcs by LB	Calcs date 19/12/2019	Checked by LB	Checked date 28/01/2020	Approved by	Approved date

TIMBER JOIST DESIGN (BS5268-2:2002)

Tedds calculation version 1.1.04

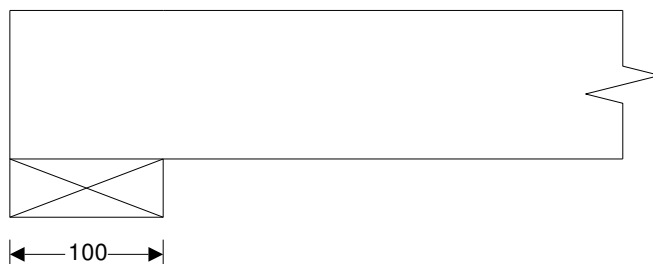
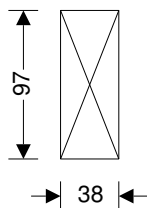
Joist details

Joist breadth	b = 38 mm
Joist depth	h = 97 mm
Joist spacing	s = 600 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} = 1100 mm



Section properties

Second moment of area	I = b × h³ / 12 = 2890131 mm⁴
Section modulus	Z = b × h² / 6 = 59590 mm³

Loading details

Joist self weight	F_{swt} = b × h × ρ_{char} × g_{acc} = 0.01 kN/m
Dead load	F_{d_udi} = 0.75 kN/m²
Imposed UDL(Medium term)	F_{i_udi} = 0.75 kN/m²
Imposed point load (Short term)	F_{i_pt} = 0.90 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	K₇ = 1.13
Load sharing factor	K₈ = 1.10

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	Flat Roof Joists				8	
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	LB	19/12/2019	LB	28/01/2020		

Consider medium term loads

Load duration factor	$K_3 = 1.25$
Maximum bending moment	$M = 0.138 \text{ kNm}$
Maximum shear force	$V = 0.502 \text{ kN}$
Maximum support reaction	$R = 0.502 \text{ kN}$
Maximum deflection	$\delta = 0.624 \text{ mm}$

Check bending stress

Bending stress	$\sigma_m = 7.500 \text{ N/mm}^2$
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 11.676 \text{ N/mm}^2$
Applied bending stress	$\sigma_{m_max} = M / Z = 2.316 \text{ N/mm}^2$

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	$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.204 \text{ N/mm}^2$

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	$\sigma_{c_max} = R / (b \times L_b) = 0.132 \text{ N/mm}^2$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection	$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = 3.300 \text{ mm}$
Bending deflection (based on E_{mean})	$\delta_{bending} = 0.557 \text{ mm}$
Shear deflection	$\delta_{shear} = 0.067 \text{ mm}$
Total deflection	$\delta = \delta_{bending} + \delta_{shear} = 0.624 \text{ mm}$

PASS - Actual deflection within permissible limits

Consider short term loads

Load duration factor	$K_3 = 1.50$
Maximum bending moment	$M = 0.317 \text{ kNm}$
Maximum shear force	$V = 1.154 \text{ kN}$
Maximum support reaction	$R = 1.154 \text{ kN}$
Maximum deflection	$\delta = 1.235 \text{ mm}$

Check bending stress

Bending stress	$\sigma_m = 7.500 \text{ N/mm}^2$
Permissible bending stress	$\sigma_{m_adm} = \sigma_m \times K_{2m} \times K_3 \times K_7 \times K_8 = 14.011 \text{ N/mm}^2$
Applied bending stress	$\sigma_{m_max} = M / Z = 5.328 \text{ N/mm}^2$

PASS - Applied bending stress within permissible limits

Check shear stress

Shear stress	$\tau = 0.710 \text{ N/mm}^2$
Permissible shear stress	$\tau_{adm} = \tau \times K_{2s} \times K_3 \times K_8 = 1.172 \text{ N/mm}^2$
Applied shear stress	$\tau_{max} = 3 \times V / (2 \times b \times h) = 0.470 \text{ N/mm}^2$

PASS - Applied shear stress within permissible limits

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	Flat Roof Joists				9	
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	LB	19/12/2019	LB	28/01/2020		

Check bearing stress

Compression perpendicular to grain (no wane)

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

Permissible bearing stress

$$\sigma_{c_adm} = \sigma_{cp1} \times K_{2c} \times K_3 \times K_8 = \mathbf{3.960 \text{ N/mm}^2}$$

Applied bearing stress

$$\sigma_{c_max} = R / (b \times L_b) = \mathbf{0.304 \text{ N/mm}^2}$$

PASS - Applied bearing stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = \min(L_{s1} \times 0.003, 14 \text{ mm}) = \mathbf{3.300 \text{ mm}}$$

Bending deflection (based on E_{mean})

$$\delta_{bending} = \mathbf{1.082 \text{ mm}}$$

Shear deflection

$$\delta_{shear} = \mathbf{0.153 \text{ mm}}$$

Total deflection

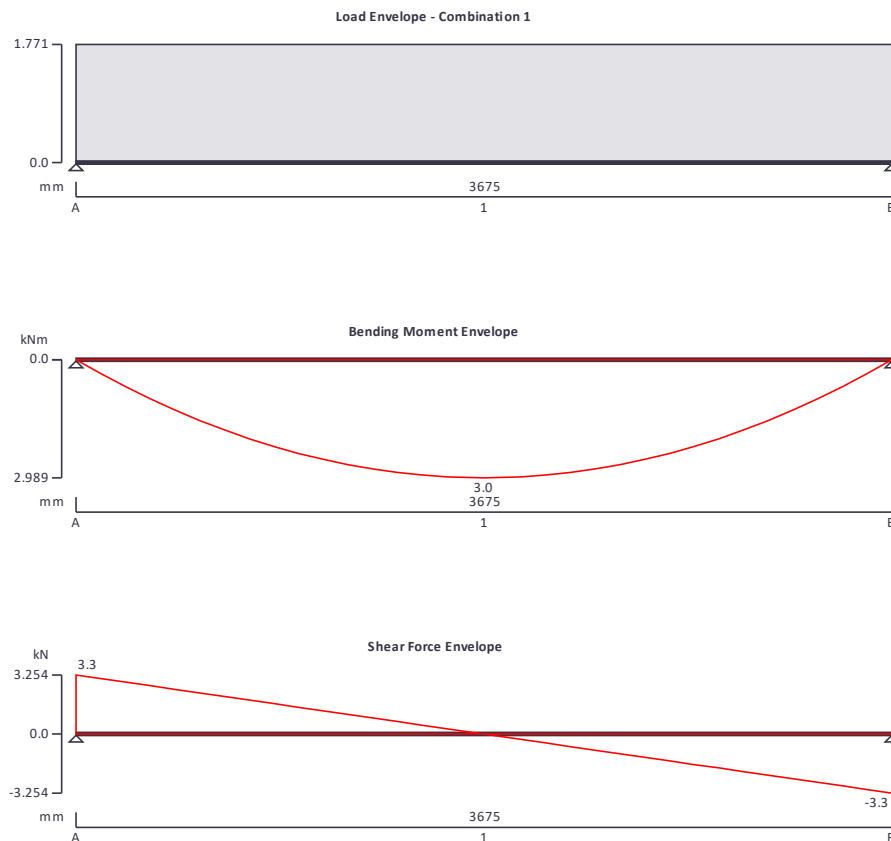
$$\delta = \delta_{bending} + \delta_{shear} = \mathbf{1.235 \text{ mm}}$$

PASS - Actual deflection within permissible limits

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
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	LB	19/12/2019	LB	28/01/2020		

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01



Applied loading

Beam loads

1M ROOF

1M ROOF

Dead self weight of beam $\times 1$

Dead full UDL 0.950 kN/m

Imposed full UDL 0.750 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.00$

Imposed $\times 1.00$

Span 1

Dead $\times 1.00$

Imposed $\times 1.00$

Support B

Dead $\times 1.00$

Imposed $\times 1.00$

Analysis results

Maximum moment

$M_{\max} = 2.989$ kNm

$M_{\min} = 0.000$ kNm

Design moment

$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 2.989$ kNm

Maximum shear

$F_{\max} = 3.254$ kN

$F_{\min} = -3.254$ kN

Design shear

$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 3.254$ kN

Total load on beam

$W_{\text{tot}} = 6.507$ kN

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Reactions at support A

$$R_{A_max} = 3.254 \text{ kN}$$

$$R_{A_min} = 3.254 \text{ kN}$$

Unfactored dead load reaction at support A

$$R_{A_Dead} = 1.875 \text{ kN}$$

Unfactored imposed load reaction at support A

$$R_{A_Imposed} = 1.378 \text{ kN}$$

Reactions at support B

$$R_{B_max} = 3.254 \text{ kN}$$

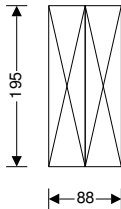
$$R_{B_min} = 3.254 \text{ kN}$$

Unfactored dead load reaction at support B

$$R_{B_Dead} = 1.875 \text{ kN}$$

Unfactored imposed load reaction at support B

$$R_{B_Imposed} = 1.378 \text{ kN}$$



Timber section details

Breadth of sections

$$b = 44 \text{ mm}$$

Depth of sections

$$h = 195 \text{ mm}$$

Number of sections in member

$$N = 2$$

Overall breadth of member

$$b_b = N \times b = 88 \text{ mm}$$

Timber strength class

$$\text{C24}$$

Member details

Service class of timber

$$1$$

Load duration

$$\text{Long term}$$

Length of span

$$L_{s1} = 3675 \text{ mm}$$

Length of bearing

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

Section properties

Cross sectional area of member

$$A = N \times b \times h = 17160 \text{ mm}^2$$

Section modulus

$$Z_x = N \times b \times h^2 / 6 = 557700 \text{ mm}^3$$

$$Z_y = h \times (N \times b)^2 / 6 = 251680 \text{ mm}^3$$

Second moment of area

$$I_x = N \times b \times h^3 / 12 = 54375750 \text{ mm}^4$$

$$I_y = h \times (N \times b)^3 / 12 = 11073920 \text{ mm}^4$$

Radius of gyration

$$i_x = \sqrt{I_x / A} = 56.3 \text{ mm}$$

$$i_y = \sqrt{I_y / A} = 25.4 \text{ mm}$$

Modification factors

Duration of loading - Table 17

$$K_3 = 1.00$$

Bearing stress - Table 18

$$K_4 = 1.00$$

Total depth of member - cl.2.10.6

$$K_7 = (300 \text{ mm} / h)^{0.11} = 1.05$$

Load sharing - cl.2.10.11

$$K_8 = 1.10$$

Minimum modulus of elasticity - Table 20

$$K_9 = 1.14$$

Lateral support - cl.2.10.8

Ends held in position

$$3.00$$

Permissible depth-to-breadth ratio - Table 19

$$h / (N \times b) = 2.22$$

Actual depth-to-breadth ratio

PASS - Lateral support is adequate

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
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	Roof Trimmer				12	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

Compression perpendicular to grain

Permissible bearing stress (no wane)

$$\sigma_{c_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_8 = \mathbf{2.640 \text{ N/mm}^2}$$

Applied bearing stress

$$\sigma_{c_a} = R_{A_max} / (N \times b \times L_b) = \mathbf{0.370 \text{ N/mm}^2}$$

$$\sigma_{c_a} / \sigma_{c_adm} = \mathbf{0.140}$$

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

Bending parallel to grain

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = \mathbf{8.650 \text{ N/mm}^2}$$

Applied bending stress

$$\sigma_{m_a} = M / Z_x = \mathbf{5.360 \text{ N/mm}^2}$$

$$\sigma_{m_a} / \sigma_{m_adm} = \mathbf{0.620}$$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress

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

Applied shear stress

$$\tau_a = 3 \times F / (2 \times A) = \mathbf{0.284 \text{ N/mm}^2}$$

$$\tau_a / \tau_{adm} = \mathbf{0.364}$$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection

$$E = E_{min} \times K_9 = \mathbf{8208 \text{ N/mm}^2}$$

Permissible deflection

$$\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) = \mathbf{11.025 \text{ mm}}$$

Bending deflection

$$\delta_{b_s1} = \mathbf{9.422 \text{ mm}}$$

Shear deflection

$$\delta_{v_s1} = \mathbf{0.407 \text{ mm}}$$

Total deflection

$$\delta_a = \delta_{b_s1} + \delta_{v_s1} = \mathbf{9.830 \text{ mm}}$$

$$\delta_a / \delta_{adm} = \mathbf{0.892}$$

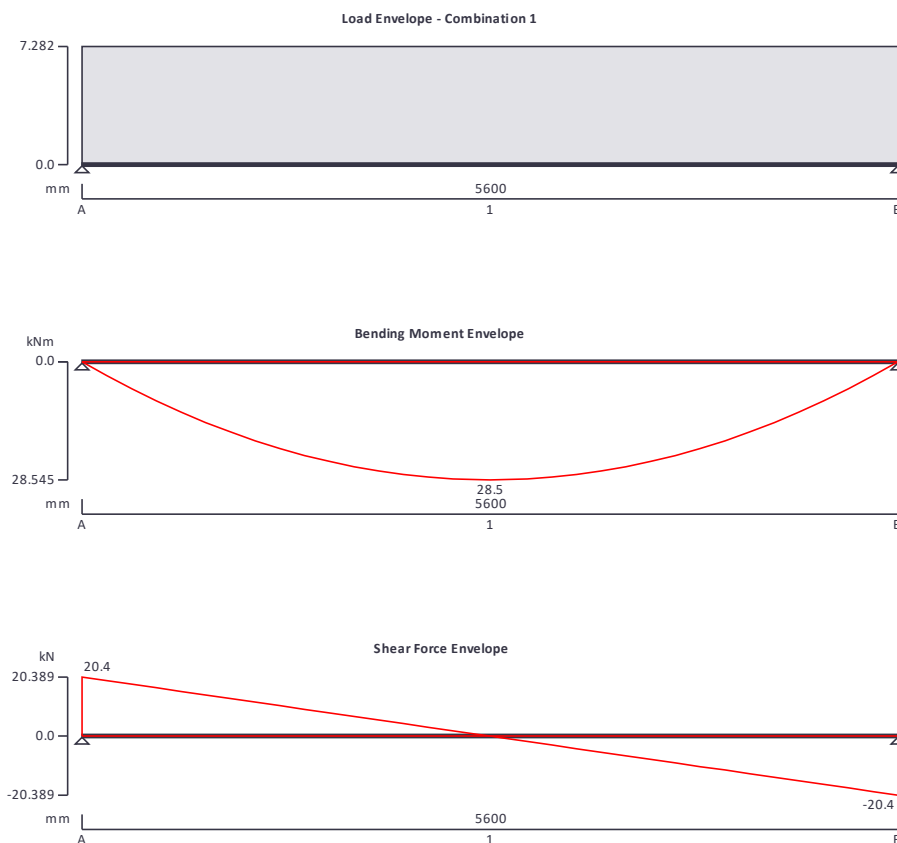
PASS - Total deflection is less than permissible deflection

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
	Calcs for				Start page no./Revision	
	Steel Purlin				13	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	28/01/2020	LB	28/01/2020		

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

3m roof - Dead full UDL 2.85 kN/m

3m roof - Imposed full UDL 1.8 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
	Calcs for				Start page no./Revision	
	Steel Purlin				14	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	28/01/2020	LB	28/01/2020		

Analysis results

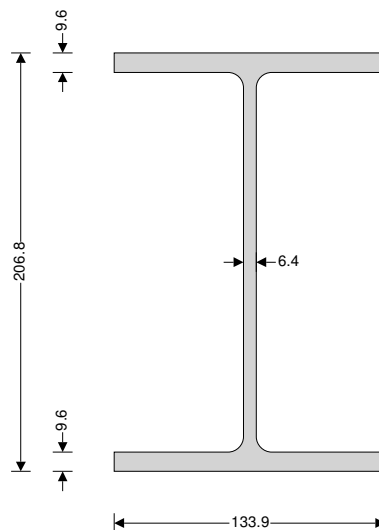
Maximum moment	$M_{\max} = 28.5 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 20.4 \text{ kN}$	$V_{\min} = -20.4 \text{ kN}$
Deflection	$\delta_{\max} = 10.7 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 20.4 \text{ kN}$	$R_{A_{\min}} = 20.4 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{\text{Dead}}} = 8.8 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{\text{Imposed}}} = 5 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 20.4 \text{ kN}$	$R_{B_{\min}} = 20.4 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{\text{Dead}}} = 8.8 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{\text{Imposed}}} = 5 \text{ kN}$	

Section details

Section type	UKB 203x133x30 (Tata Steel Advance)
Steel grade	S275

From table 9: Design strength p_y

Thickness of element	$\max(T, t) = 9.6 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has 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	$K_{LT,A} = 1.20$
	$K_{LT,B} = 1.00$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section	$d = 172.4 \text{ mm}$	
	$d / t = 26.9 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
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	Steel Purlin				15	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	28/01/2020	LB	28/01/2020		

Outstand flanges - Table 11

Width of section

$$b = B / 2 = 67 \text{ mm}$$

$$b / T = 7.0 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 20.4 \text{ kN}$$

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

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1324 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 218.4 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 28.5 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 86.5 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 1.2 \times L_{s1} = 6720 \text{ mm}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = 211.801$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.881$$

Torsional index

$$x = 21.493$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.643$$

Ratio - cl.4.3.6.9

$$\beta_w = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 120.018$$

Limiting slenderness - Annex B.2.2

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

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

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = 7.0$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.600$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 140.5 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 249.9 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 95.6 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB

$$m_{LT} = 1.000$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = 30 \text{ kNm}$$

$$M_b / m_{LT} = 30 \text{ kNm}$$

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

$$\delta_{lim} = L_{s1} / 360 = 15.556 \text{ mm}$$

Maximum deflection span 1

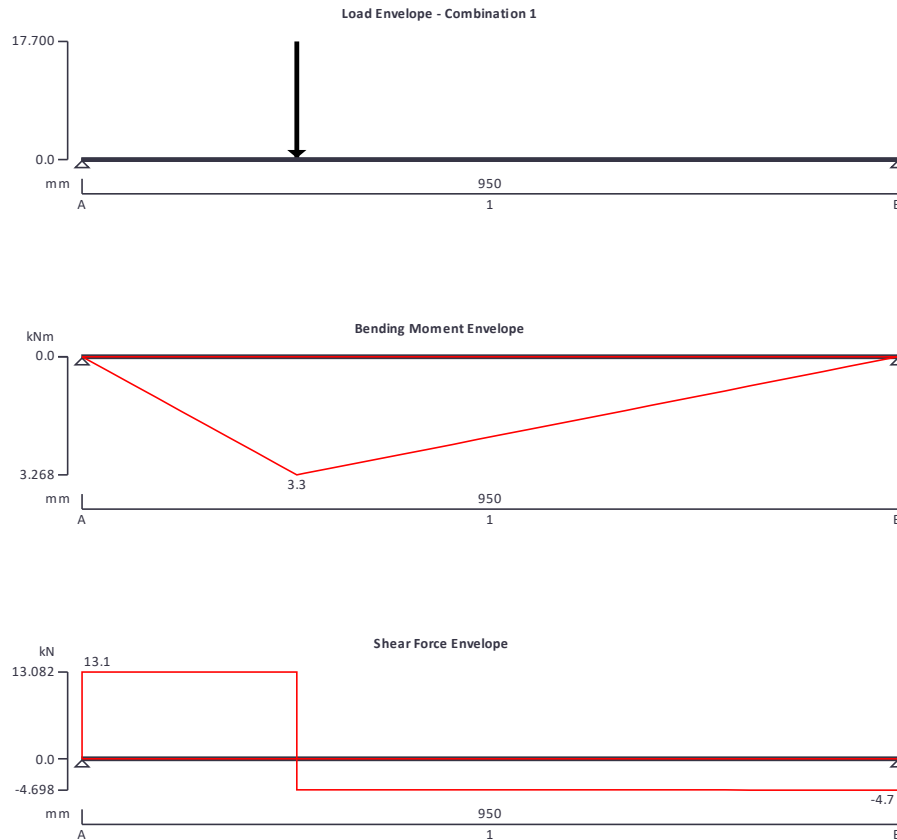
$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 10.666 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

Project				Job no.	
Gethin House				LS378	
Calcs for				Start page no./Revision	
L2				16	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
LB	19/12/2019	LB	28/01/2020		

GLULAM BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.01



Applied loading

Beam loads

RIDGE BEAM

RIDGE BEAM

Dead self weight of beam $\times 1$

Dead point load 10.000 kN at 250 mm

Imposed point load 7.700 kN at 250 mm

Load combinations

Load combination 1

Support A

Dead $\times 1.00$

Imposed $\times 1.00$

Span 1

Dead $\times 1.00$

Imposed $\times 1.00$

Support B

Dead $\times 1.00$

Imposed $\times 1.00$

Analysis results

Maximum moment

$M_{\max} = 3.268$ kNm

$M_{\min} = 0.000$ kNm

Design moment

$M = \max(\text{abs}(M_{\max}), \text{abs}(M_{\min})) = 3.268$ kNm

Maximum shear

$F_{\max} = 13.082$ kN

$F_{\min} = -4.698$ kN

Design shear

$F = \max(\text{abs}(F_{\max}), \text{abs}(F_{\min})) = 13.082$ kN

Total load on beam

$W_{\text{tot}} = 17.779$ kN

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
	Calcs for				Start page no./Revision	
	L2				17	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

Reactions at support A

$$R_{A_max} = 13.082 \text{ kN}$$

$$R_{A_min} = 13.082 \text{ kN}$$

Unfactored dead load reaction at support A

$$R_{A_Dead} = 7.408 \text{ kN}$$

Unfactored imposed load reaction at support A

$$R_{A_Imposed} = 5.674 \text{ kN}$$

Reactions at support B

$$R_{B_max} = 4.698 \text{ kN}$$

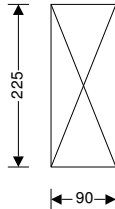
$$R_{B_min} = 4.698 \text{ kN}$$

Unfactored dead load reaction at support B

$$R_{B_Dead} = 2.671 \text{ kN}$$

Unfactored imposed load reaction at support B

$$R_{B_Imposed} = 2.026 \text{ kN}$$



Glulam section details

Breadth of section

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

Depth of section

$$h = 225 \text{ mm}$$

Number of sections in member

$$N = 1$$

Overall breadth of member

$$b_b = N \times b = 90 \text{ mm}$$

Number of laminations

$$N_{lam} = 5$$

Alignment of laminations

Horizontal

Timber strength class

C24

Member details

Service class of timber

1

Load duration

Medium term

Length of span

$$L_{s1} = 950 \text{ mm}$$

Length of bearing

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

Section properties

Cross sectional area of member

$$A = N \times b \times h = 20250 \text{ mm}^2$$

Section modulus

$$Z_x = N \times b \times h^2 / 6 = 759375 \text{ mm}^3$$

$$Z_y = h \times (N \times b)^2 / 6 = 303750 \text{ mm}^3$$

Second moment of area

$$I_x = N \times b \times h^3 / 12 = 85429687 \text{ mm}^4$$

$$I_y = h \times (N \times b)^3 / 12 = 13668750 \text{ mm}^4$$

Radius of gyration

$$i_x = \sqrt{I_x / A} = 65.0 \text{ mm}$$

$$i_y = \sqrt{I_y / A} = 26.0 \text{ mm}$$

Modification factors

Duration of loading - Table 17

$$K_3 = 1.25$$

Bearing stress - Table 18

$$K_4 = 1.00$$

Total depth of member - cl.2.10.6

$$K_7 = (300 \text{ mm} / h)^{0.11} = 1.03$$

Bending parallel to grain - Table 24

$$K_{15} = 1.34$$

Tension parallel to grain - Table 24

$$K_{16} = 1.34$$

Compression parallel to grain - Table 24

$$K_{17} = 1.04$$

Compression perpendicular to grain - Table 24

$$K_{18} = 1.55$$

Shear parallel to grain - Table 24

$$K_{19} = 2.34$$

Modulus of elasticity - Table 24

$$K_{20} = 1.07$$

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
	Calcs for				Start page no./Revision	
	L2				18	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

Lateral support - cl.2.10.8

Ends held in position

Permissible depth-to-breadth ratio - Table 19

3.00

Actual depth-to-breadth ratio

$h / (N \times b) = 2.50$

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress

$\sigma_{c_adm} = \sigma_{cp1} \times K_3 \times K_4 \times K_{18} = 4.650 \text{ N/mm}^2$

Applied bearing stress

$\sigma_{c_a} = R_{A_max} / (N \times b \times L_b) = 1.454 \text{ N/mm}^2$

$\sigma_{c_a} / \sigma_{c_adm} = 0.313$

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

Bending parallel to grain

Permissible bending stress

$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_{15} = 12.966 \text{ N/mm}^2$

Applied bending stress

$\sigma_{m_a} = M / Z_x = 4.303 \text{ N/mm}^2$

$\sigma_{m_a} / \sigma_{m_adm} = 0.332$

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress

$\tau_{adm} = \tau \times K_3 \times K_{19} = 2.077 \text{ N/mm}^2$

Applied shear stress

$\tau_a = 3 \times F / (2 \times A) = 0.969 \text{ N/mm}^2$

$\tau_a / \tau_{adm} = 0.467$

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection

$E = E_{mean} \times K_{20} = 11556 \text{ N/mm}^2$

Permissible deflection

$\delta_{adm} = \min(0.551 \text{ in}, 0.003 \times L_{s1}) = 2.850 \text{ mm}$

Bending deflection

$\delta_{b_s1} = 0.234 \text{ mm}$

Shear deflection

$\delta_{v_s1} = 0.268 \text{ mm}$

Total deflection

$\delta_a = \delta_{b_s1} + \delta_{v_s1} = 0.502 \text{ mm}$

$\delta_a / \delta_{adm} = 0.176$

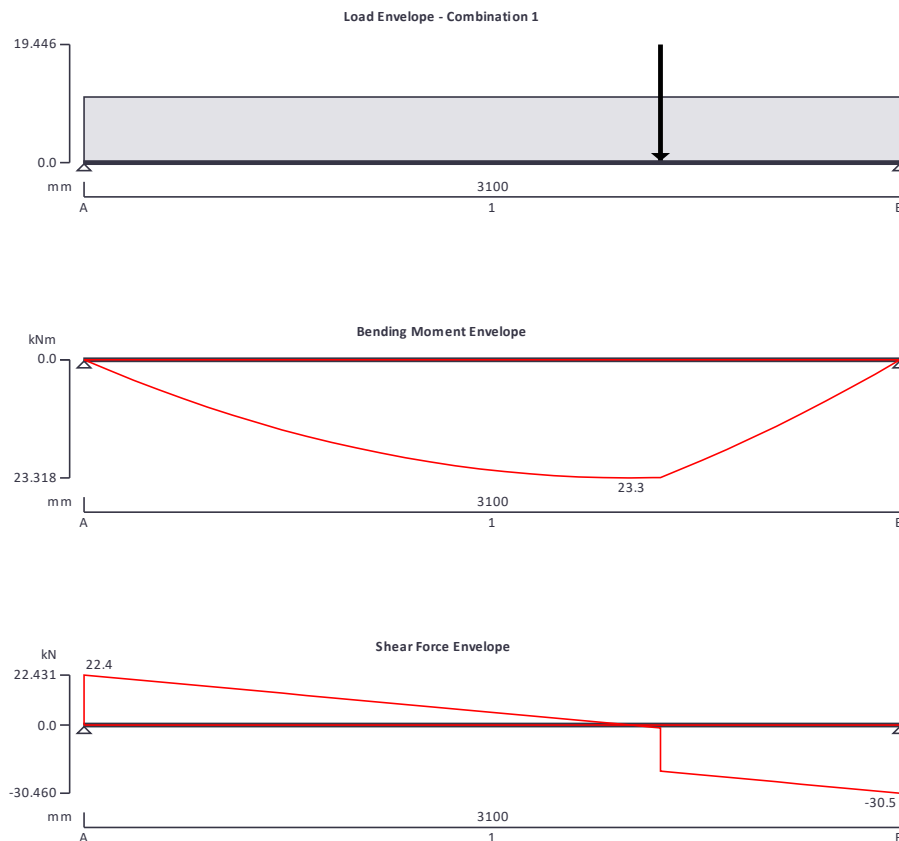
PASS - Total deflection is less than permissible deflection

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
	Calcs for				Start page no./Revision	
	Beam A				19	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

L2 - Dead point load 7.41 kN at 2190 mm

L2 - Imposed point load 5.67 kN at 2190 mm

STUD + 2.8M FLOOR - Dead full UDL 2.68 kN/m

2.8M FLOOR - Imposed full UDL 4.2 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
	Calcs for				Start page no./Revision	
	Beam A				20	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

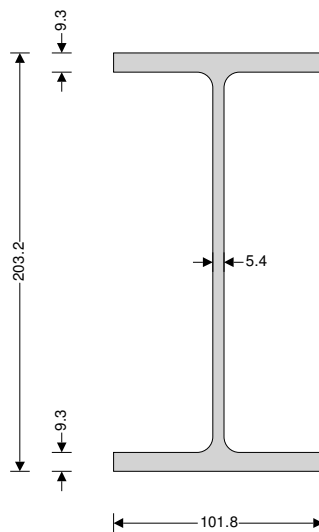
Imposed $\times 1.60$

Analysis results

Maximum moment	$M_{\max} = 23.3 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 22.4 \text{ kN}$	$V_{\min} = -30.5 \text{ kN}$
Deflection	$\delta_{\max} = 3.5 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 22.4 \text{ kN}$	$R_{A_{\min}} = 22.4 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{\text{Dead}}} = 6.7 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{\text{Imposed}}} = 8.2 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 30.5 \text{ kN}$	$R_{B_{\min}} = 30.5 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{\text{Dead}}} = 9.7 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{\text{Imposed}}} = 10.5 \text{ kN}$	

Section details

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



Lateral restraint

Span 1 has 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	$K_{LTA} = 1.20 + 2 \times D$
	$K_{LTB} = 1.20 + 2 \times D$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section	$d = 169.4 \text{ mm}$	
	$d / t = 31.4 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
	Calcs for				Start page no./Revision	
	Beam A				21	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

Outstand flanges - Table 11

Width of section

$$b = B / 2 = 50.9 \text{ mm}$$

$$b / T = 5.5 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 30.5 \text{ kN}$$

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

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1097 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 181.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 23.3 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 64.4 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 1.2 \times L_{s1} + 2 \times D = 4126 \text{ mm}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = 174.779$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.888$$

Torsional index

$$x = 22.460$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.706$$

Ratio - cl.4.3.6.9

$$\beta_w = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 109.560$$

Limiting slenderness - Annex B.2.2

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

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

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = 7.0$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.527$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 168.6 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 266.2 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 109.7 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB

$$m_{LT} = 1.000$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = 25.7 \text{ kNm}$$

$$M_b / m_{LT} = 25.7 \text{ kNm}$$

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

$$\delta_{lim} = L_{s1} / 360 = 8.611 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 3.455 \text{ mm}$$

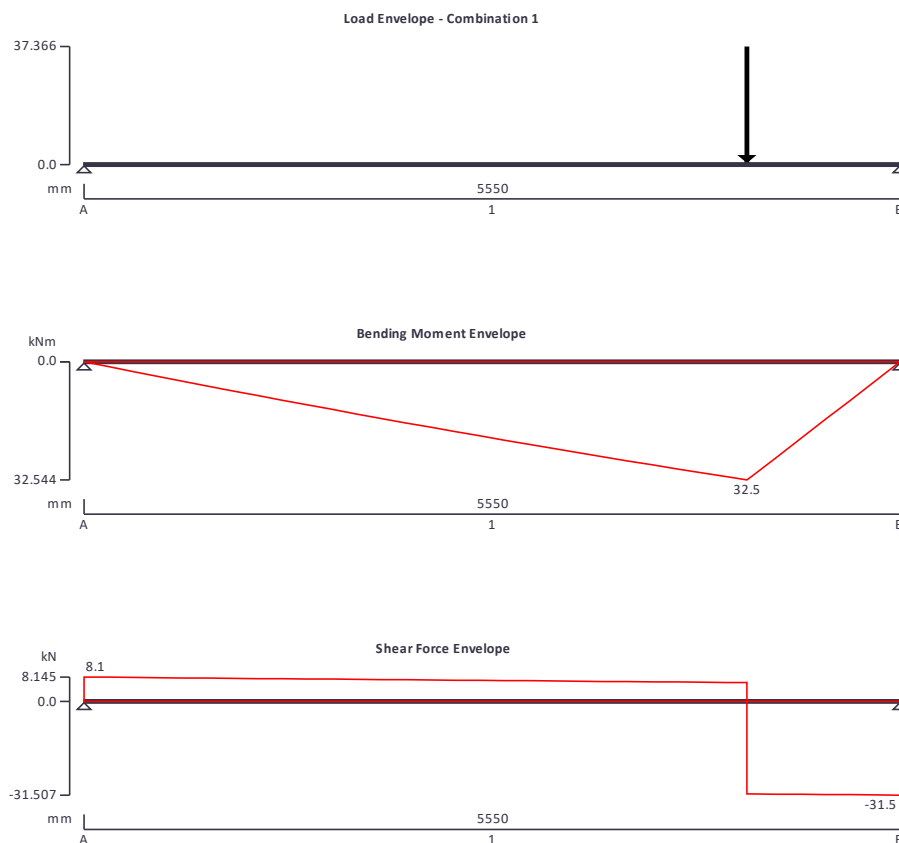
PASS - Maximum deflection does not exceed deflection limit

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
	Calcs for				Start page no./Revision	
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	LB	19/12/2019	LB	28/01/2020		

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

L2 & BEAM A - Dead point load 12.37 kN at 4510 mm

L2 & BEAM A - Imposed point load 12.53 kN at 4510 mm

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 $\times 1.60$

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
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	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

Analysis results

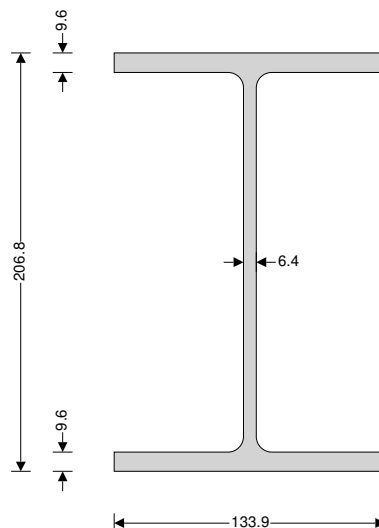
Maximum moment	$M_{\max} = 32.5 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 8.1 \text{ kN}$	$V_{\min} = -31.5 \text{ kN}$
Deflection	$\delta_{\max} = 8.8 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 8.1 \text{ kN}$	$R_{A_{\min}} = 8.1 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{\text{Dead}}} = 3.1 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{\text{Imposed}}} = 2.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 31.5 \text{ kN}$	$R_{B_{\min}} = 31.5 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{\text{Dead}}} = 10.9 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{\text{Imposed}}} = 10.2 \text{ kN}$	

Section details

Section type	UB 203x133x30 (BS4-1)
Steel grade	S275

From table 9: Design strength p_y

Thickness of element	$\max(T, t) = 9.6 \text{ mm}$
Design strength	$p_y = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$



Lateral restraint

Span 1 has 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	$K_{LT,A} = 0.95$
	$K_{LT,B} = 0.95$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section	$d = 172.4 \text{ mm}$	
	$d / t = 26.9 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

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Outstand flanges - Table 11

Width of section

$$b = B / 2 = 67 \text{ mm}$$

$$b / T = 7.0 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 31.5 \text{ kN}$$

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

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1324 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 218.4 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 32.5 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 86.5 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 0.95 \times L_{s1} = 5273 \text{ mm}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = 166.179$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.881$$

Torsional index

$$x = 21.493$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.708$$

Ratio - cl.4.3.6.9

$$\beta_w = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 103.650$$

Limiting slenderness - Annex B.2.2

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

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

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = 7.0$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.485$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 188.3 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 277.4 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 118.8 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB

$$m_{LT} = 1.000$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = 37.3 \text{ kNm}$$

$$M_b / m_{LT} = 37.3 \text{ kNm}$$

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

$$\delta_{lim} = L_{s1} / 360 = 15.417 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 8.771 \text{ mm}$$

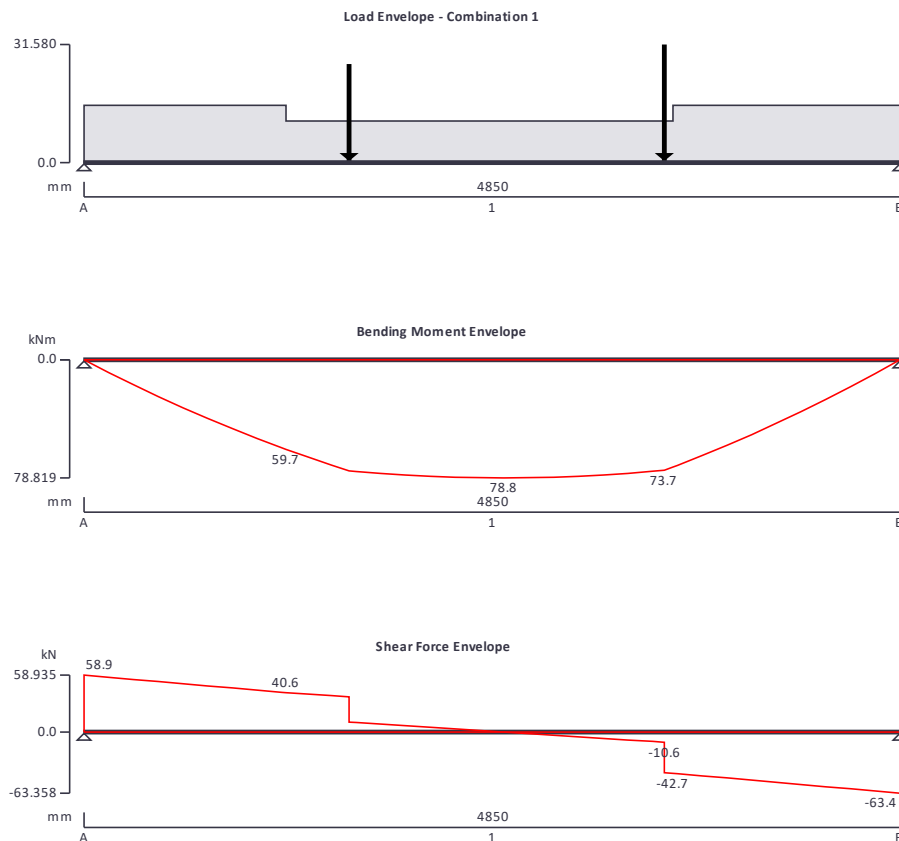
PASS - Maximum deflection does not exceed deflection limit

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
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	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

PURLIN - Dead point load 10 kN at 1575 mm

PURLIN - Imposed point load 7.7 kN at 1575 mm

BEAM B - Dead point load 10.9 kN at 3450 mm

BEAM B - Imposed point load 10.2 kN at 3450 mm

STUD PANEL + 2.8M FLOOR - Dead full UDL 2.68 kN/m

2.8M FLOOR - Imposed full UDL 4.2 kN/m

1.3M FLOOR - Dead partial UDL 0.78 kN/m from 0 mm to 1200 mm

1.3M FLOOR - Imposed partial UDL 1.95 kN/m from 0 mm to 1200 mm

mm

1.3M FLOOR - Dead partial UDL 0.78 kN/m from 3500 mm to 4850 mm

mm

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
	Calcs for				Start page no./Revision	
	Beam C				26	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

1.3M FLOOR - Imposed partial UDL 1.95 kN/m from 3500 mm to 4850 mm

Load combinations

Load combination 1

Support A

Dead $\times$ 1.40

Imposed $\times$ 1.60

Dead $\times$ 1.40

Imposed $\times$ 1.60

Support B

Dead $\times$ 1.40

Imposed $\times$ 1.60

Analysis results

Maximum moment

$M_{\max} = 78.8$ kNm

$M_{\min} = 0$ kNm

Maximum shear

$V_{\max} = 58.9$ kN

$V_{\min} = -63.4$ kN

Deflection

$\delta_{\max} = 9.9$ mm

$\delta_{\min} = 0$ mm

Maximum reaction at support A

$R_{A_{\max}} = 58.9$ kN

$R_{A_{\min}} = 58.9$ kN

Unfactored dead load reaction at support A

$R_{A_{\text{Dead}}} = 18.4$ kN

Unfactored imposed load reaction at support A

$R_{A_{\text{Imposed}}} = 20.7$ kN

Maximum reaction at support B

$R_{B_{\max}} = 63.4$ kN

$R_{B_{\min}} = 63.4$ kN

Unfactored dead load reaction at support B

$R_{B_{\text{Dead}}} = 19.5$ kN

Unfactored imposed load reaction at support B

$R_{B_{\text{Imposed}}} = 22.5$ kN

Section details

Section type

UB 254x146x43 (BS4-1)

Steel grade

S275

From table 9: Design strength p_y

Thickness of element

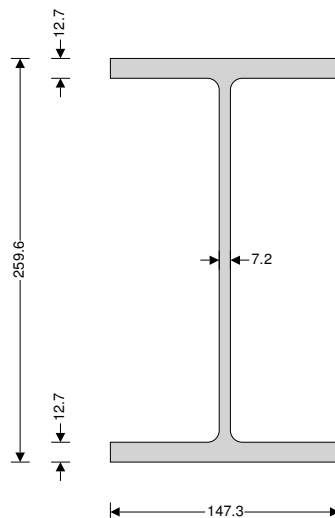
$\max(T, t) = 12.7$ mm

Design strength

$p_y = 275$ 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

$K_y = 1.00$

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	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

Effective length factor for lateral-torsional buckling $K_{LT,A} = 0.95$

$K_{LT,B} = 0.95$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section

$$d = 219 \text{ mm}$$

$$d / t = 30.4 \times \varepsilon \leq 80 \times \varepsilon \quad \text{Class 1 plastic}$$

Outstand flanges - Table 11

Width of section

$$b = B / 2 = 73.7 \text{ mm}$$

$$b / T = 5.8 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1 plastic}$$

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 63.4 \text{ kN}$$

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

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1869 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 308.4 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 78.8 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 155.7 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 0.95 \times L_{s1} = 4608 \text{ mm}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = 131.019$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.891$$

Torsional index

$$x = 21.162$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^{2.25}] = 0.765$$

Ratio - cl.4.3.6.9

$$\beta_w = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 89.282$$

Limiting slenderness - Annex B.2.2

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

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

Bending strength - Section 4.3.6.5

Robertson constant

$$\alpha_{LT} = 7.0$$

Perry factor

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.385$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 253.8 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 313.2 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 145 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB

$$m_{LT} = 1.000$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = 82.1 \text{ kNm}$$

$$M_b / m_{LT} = 82.1 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
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Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{13.472 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{9.891 \text{ mm}}$$

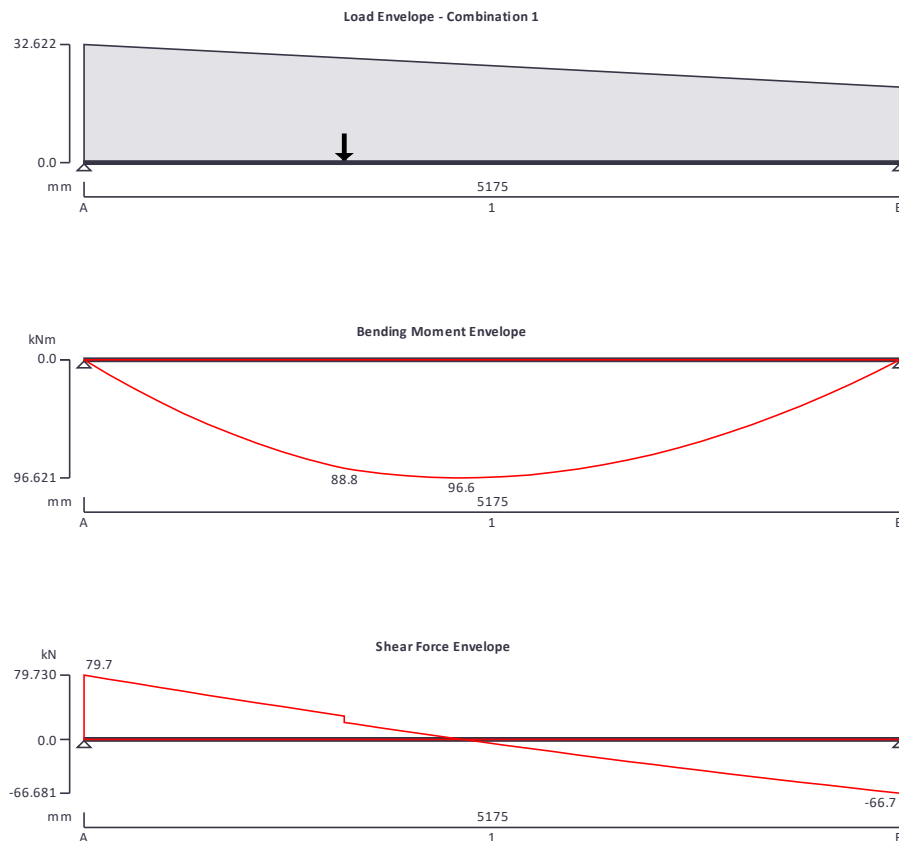
PASS - Maximum deflection does not exceed deflection limit

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
	Calcs for				Start page no./Revision	
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STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

BEAM B - Dead point load 3.1 kN at 1650 mm

BEAM B - Imposed point load 2.3 kN at 1650 mm

2.8M FLOOR + PURLIN - Dead full UDL 4.48 kN/m

2.8M FLOOR + PURLIN - Imposed full UDL 5.6 kN/m

MASONRY - 5M $\rightarrow$ 1.5M - Dead full VDL 12 kN/m to 3.6 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

Imposed $\times 1.60$

Analysis results

Maximum moment

$M_{\max} = 96.6 \text{ kNm}$

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

Maximum moment span 1 segment 1

$M_{s1_seg1_max} = 88.8 \text{ kNm}$

$M_{s1_seg1_min} = 0 \text{ kNm}$

Maximum moment span 1 segment 2

$M_{s1_seg2_max} = 96.6 \text{ kNm}$

$M_{s1_seg2_min} = 0 \text{ kNm}$

Maximum shear

$V_{\max} = 79.7 \text{ kN}$

$V_{\min} = -66.7 \text{ kN}$

Maximum shear span 1 segment 1

$V_{s1_seg1_max} = 79.7 \text{ kN}$

$V_{s1_seg1_min} = 0 \text{ kN}$

Maximum shear span 1 segment 2

$V_{s1_seg2_max} = 21 \text{ kN}$

$V_{s1_seg2_min} = -66.7 \text{ kN}$

Deflection segment 3

$\delta_{\max} = 13.7 \text{ mm}$

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

Maximum reaction at support A

$R_{A_max} = 79.7 \text{ kN}$

$R_{A_min} = 79.7 \text{ kN}$

Unfactored dead load reaction at support A

$R_{A_Dead} = 38.6 \text{ kN}$

Unfactored imposed load reaction at support A

$R_{A_Imposed} = 16.1 \text{ kN}$

Maximum reaction at support B

$R_{B_max} = 66.7 \text{ kN}$

$R_{B_min} = 66.7 \text{ kN}$

Unfactored dead load reaction at support B

$R_{B_Dead} = 30.2 \text{ kN}$

Unfactored imposed load reaction at support B

$R_{B_Imposed} = 15.2 \text{ kN}$

Section details

Section type

UB 254x146x43 (BS4-1)

Steel grade

S355

From table 9: Design strength p_y

Thickness of element

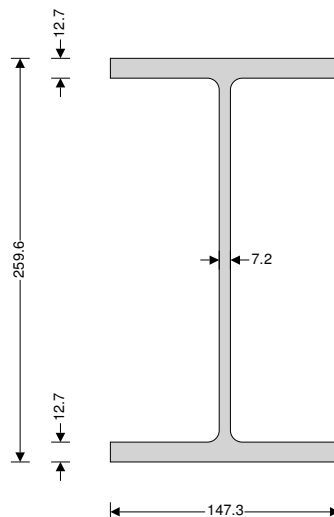
$\max(T, t) = 12.7 \text{ mm}$

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 plus 1650 mm

Effective length factors

Effective length factor in major axis

$K_x = 1.00$

Effective length factor in minor axis

$K_y = 1.00$

Effective length factor for lateral-torsional buckling

$K_{LT,A} = 0.95$

$K_{LT,B} = 0.95$

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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

$$d = \mathbf{219 \text{ mm}}$$

$$d / t = 34.6 \times \varepsilon \leq 80 \times \varepsilon$$

Class 1 plastic

Outstand flanges - Table 11

Width of section

$$b = B / 2 = \mathbf{73.7 \text{ mm}}$$

$$b / T = 6.6 \times \varepsilon \leq 9 \times \varepsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{79.7 \text{ kN}}$$

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

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = \mathbf{1869 \text{ mm}^2}$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = \mathbf{398.1 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 segment 2 - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_seg2_max}), \text{abs}(M_{s1_seg2_min})) = \mathbf{96.6 \text{ kNm}}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{201 \text{ kNm}}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = (1.2 + 0.95) \times L_{s1_seg2} / 2 = \mathbf{3789 \text{ mm}}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = \mathbf{107.755}$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = \mathbf{0.891}$$

Torsional index

$$x = \mathbf{21.162}$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^{2.25}] = \mathbf{0.812}$$

Ratio - cl.4.3.6.9

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

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = \mathbf{77.951}$$

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

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.334}$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{333 \text{ N/mm}^2}$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{399.6 \text{ N/mm}^2}$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{195.9 \text{ N/mm}^2}$$

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = \mathbf{110.9 \text{ kNm}}$$

$$M_b / m_{LT} = \mathbf{110.9 \text{ kNm}}$$

PASS - Buckling resistance moment exceeds design bending moment

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
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	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{14.375 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{13.706 \text{ mm}}$$

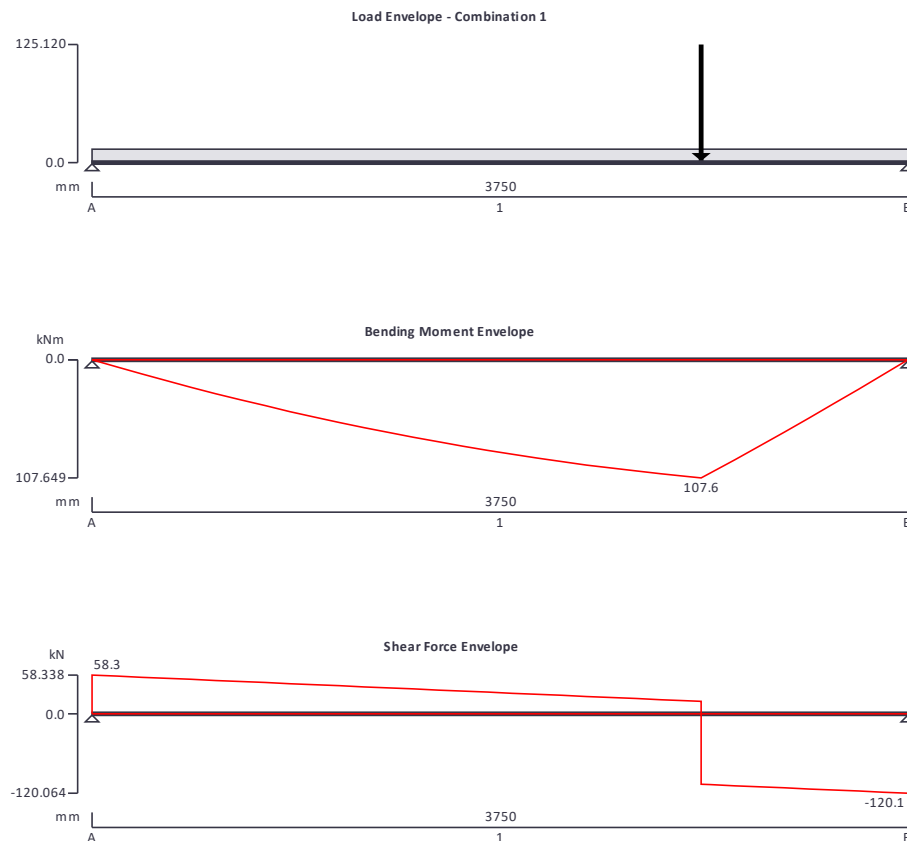
PASS - Maximum deflection does not exceed deflection limit

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
	Calcs for				Start page no./Revision	
	Beam E				33	
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STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Dead self weight of beam $\times 1$

Dead point load 58.4 kN at 2800 mm

Imposed point load 27.1 kN at 2800 mm

Dead full UDL 8.87 kN/m

Imposed full UDL 0.75 kN/m

Load combinations

Load combination 1

Support A

Dead $\times 1.40$

Imposed $\times 1.60$

Dead $\times 1.40$

Imposed $\times 1.60$

Support B

Dead $\times 1.40$

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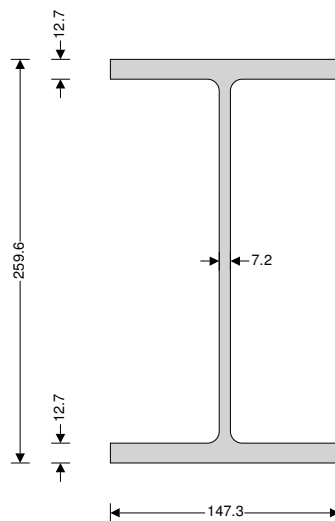
Imposed $\times 1.60$

Analysis results

Maximum moment	$M_{\max} = 107.6 \text{ kNm}$	$M_{\min} = 0 \text{ kNm}$
Maximum shear	$V_{\max} = 58.3 \text{ kN}$	$V_{\min} = -120.1 \text{ kN}$
Deflection	$\delta_{\max} = 6.9 \text{ mm}$	$\delta_{\min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_{\max}} = 58.3 \text{ kN}$	$R_{A_{\min}} = 58.3 \text{ kN}$
Unfactored dead load reaction at support A	$R_{A_{\text{Dead}}} = 32.2 \text{ kN}$	
Unfactored imposed load reaction at support A	$R_{A_{\text{Imposed}}} = 8.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_{\max}} = 120.1 \text{ kN}$	$R_{B_{\min}} = 120.1 \text{ kN}$
Unfactored dead load reaction at support B	$R_{B_{\text{Dead}}} = 61 \text{ kN}$	
Unfactored imposed load reaction at support B	$R_{B_{\text{Imposed}}} = 21.6 \text{ kN}$	

Section details

Section type	UKB 254x146x43 (Tata Steel Advance)
Steel grade	S355
From table 9: Design strength p_y	
Thickness of element	$\max(T, t) = 12.7 \text{ mm}$
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	$K_{LT,A} = 0.95$
	$K_{LT,B} = 0.95$

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	$d = 219 \text{ mm}$	
	$d / t = 34.6 \times \varepsilon \leq 80 \times \varepsilon$	Class 1 plastic

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Outstand flanges - Table 11

Width of section

$$b = B / 2 = 73.7 \text{ mm}$$

$$b / T = 6.6 \times \epsilon \leq 9 \times \epsilon$$

Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force

$$F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 120.1 \text{ kN}$$

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

Web does not need to be checked for shear buckling

Shear area

$$A_v = t \times D = 1869 \text{ mm}^2$$

Design shear resistance

$$P_v = 0.6 \times p_y \times A_v = 398.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 107.6 \text{ kNm}$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = 201 \text{ kNm}$$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling

$$L_E = 0.95 \times L_{s1} = 3563 \text{ mm}$$

Slenderness ratio

$$\lambda = L_E / r_{yy} = 101.303$$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter

$$u = 0.891$$

Torsional index

$$x = 21.162$$

Slenderness factor

$$v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = 0.826$$

Ratio - cl.4.3.6.9

$$\beta_w = 1.000$$

Equivalent slenderness - cl.4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = 74.537$$

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

$$\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = 0.310$$

Euler stress

$$p_E = \pi^2 \times E / \lambda_{LT}^2 = 364.2 \text{ N/mm}^2$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = 416.1 \text{ N/mm}^2$$

Bending strength - Annex B.2.1

$$p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = 206.7 \text{ N/mm}^2$$

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB

$$m_{LT} = 1.000$$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment

$$M_b = p_b \times S_{xx} = 117 \text{ kNm}$$

$$M_b / m_{LT} = 117 \text{ kNm}$$

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

$$\delta_{lim} = L_{s1} / 360 = 10.417 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 6.851 \text{ mm}$$

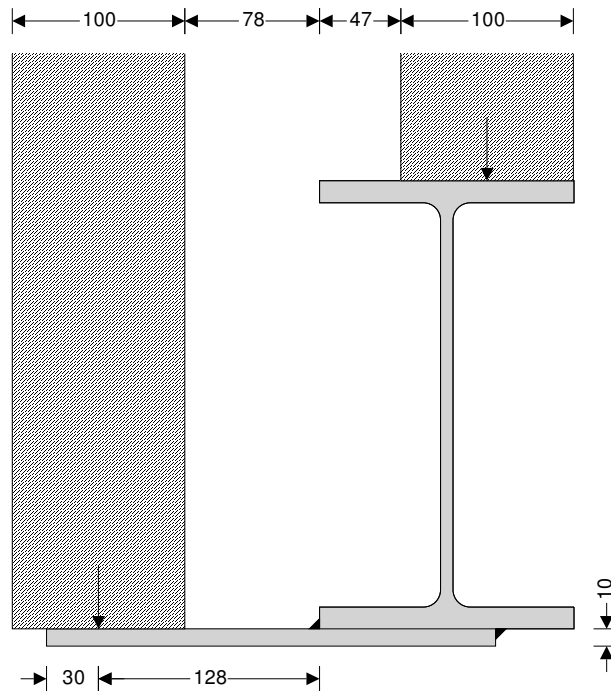
PASS - Maximum deflection does not exceed deflection limit

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STEEL MASONRY SUPPORT

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

Tedds calculation version 1.0.04



Steel member details

Torsion beam	UB 254x146x43
Masonry support plate	User
Steel grade of support plate	S275
Design strength of support plate	$p_{ysb} = 275 \text{ N/mm}^2$
Modulus of elasticity	$E = 205000 \text{ N/mm}^2$
Constant	$\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_{ysb})} = 1.000$
Length of plate beyond beam	$l_h = 158 \text{ mm}$
Total length of plate	$l_{plate} = 260 \text{ mm}$
Thickness of plate	$t_{sb} = 10 \text{ mm}$
Width of main beam	$B_{mb} = 147 \text{ mm}$
Area of plate	$A_{sbu} = t_{sb} \times l_{plate} = 2600.0 \text{ mm}^2$
Distance from weld position to CoG	$c_{yysb} = l_h / 2 - (l_{plate} - l_h) / 2 = 28 \text{ mm}$

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	$h_{mmb} = 2000 \text{ mm}$
Eccentricity of main beam material	$e_{mb} = 47 \text{ mm}$
Add dead force main beam (not from masonry)	$P_{Gaddmb} = 1.0 \text{ kN/m}$
Add live force main beam (not from masonry)	$P_{Qaddmb} = 0.8 \text{ kN/m}$
Density of masonry on support beam	$\rho_{m,sb} = 20.0 \text{ kN/m}^3$
Width of masonry on support beam	$b_{msb} = 100 \text{ mm}$
Height of masonry on support beam	$h_{msb} = 2000 \text{ mm}$

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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 $c = 125$ 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 $M_x = 25.4$ kNm

Dist to NA combined section (CoG torsion beam) $y_{e,all} = (D_{mb} + t_{sb}) \times A_{sbu} / (2 \times (A_{mb} + A_{sbu})) = 43$ mm

Second moment of area of combined section $I_{xx,all} = (I_{xxmb} + A_{mb} \times y_{e,all}^2) + A_{sbu} \times (D_{mb} / 2 + t_{sb} / 2 - y_{e,all})^2 = 9747$ cm⁴

Elastic section modulus of combined section $Z_{xx,all} = I_{xx,all} / (D_{mb} / 2 + t_{sb} - y_{e,all}) = 1011.05$ cm³

Section modulus of plate $Z_{xx,plate} = 1m \times t_{sb}^2 / (6 \times 1m) = 16.67$ cm³/m

Eccentricity of support beam masonry $e_1 = 128$ mm

Force of masonry on support plate $P_1 = (b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ} = 5.6$ kN/m

Bending at heel $M_{x,plate} = P_1 \times e_1 = 0.7$ kNm/m

Moment capacity of plate $M_c = 1.2 \times Z_{x,plate} \times p_{ysb} = 5.5$ kNm/m

PASS - Design strength exceeds stress at heel

Longitudinal stress due to overall bending $\sigma_1 = M_x / Z_{xx,all} = 25.1$ N/mm²

Constant relating to Von Mises curve $C_{fp} = (4 \times p_{ysb}^2 - 3 \times \sigma_1^2)^{0.5} = 548.3$ N/mm²

Transverse bending stress ratio limit $\alpha_{ts} = (C_{fp}^2 - \sigma_1^2) / (2 \times C_{fp} \times p_{ysb}) = 0.995$

Transverse bending stress ratio $\alpha_{ts} = M_{x,plate} / M_c = 0.130$

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfactored force on support angle $P_{1SLS} = b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb} = 4.0$ kN/m

Distance from weld to load position $a_m = e_1 = 128$ mm

Length of load resultant to edge of plate $b_m = l_h - e_1 = 30$ mm

Dist from weld to load position as ratio of length $a_l = a_m / (a_m + b_m) = 0.810$

Effective second moment of inertia $I_{eff_def} = t_{sb}^3 / 12 = 83333$ mm⁴/m

Deflection at toe $\delta = (a_l^2 \times (3 - a_l) / 6) \times (P_{1SLS} \times (a_m + b_m)^3) / (E_{S5950} \times I_{eff_def}) = 0.22$ mm

Deflection limit $\delta_{lim} = 1.64$ 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 $R_A = P_1 \times \max((1 + (3 \times e_1) / (2 \times B_{mb} / 2)), 1.4) = 20.2$ kN/m

Maximum possible force in plate $R_p = (l_h + B_{mb}) \times t_{sb} \times p_{ysb} = 839.6$ kN

Longitudinal shear between beam and plate $R_l = 2 \times R_p / L = 450.8$ kN/m

Horizontal shear between beam and plate $R_h = P_1 \times e_1 / (s_{weld} / 2 + t_{sb} / 2) = 89.6$ kN/m

Resultant weld force $R_{weld} = (R_A^2 + R_l^2 + R_h^2)^{0.5} = 0.460$ 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 $w_{1ULS} = (h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ} = 5.60$ kN/m

Loading of main beam masonry $w_{2ULS} = (h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb}) \times \gamma_{fG} + P_{Qaddmb} \times \gamma_{fQ} = 8.13$ kN/m

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Self weight of support beam

$$W_{3ULS} = A_{sbu} \times \rho_{sb} \times \gamma_f G = \mathbf{0.29 \text{ kN/m}}$$

Torsional loading SLS

Loading of support beam masonry

$$W_{1SLS} = h_{msb} \times b_{msb} \times \rho_{m, sb} + P_{Gaddsb} + P_{Qaddsb} = \mathbf{4.00 \text{ kN/m}}$$

Loading of main beam masonry

$$W_{2SLS} = h_{mmb} \times b_{mmb} \times \rho_{m, mb} + P_{Gaddmb} + P_{Qaddmb} = \mathbf{5.70 \text{ kN/m}}$$

Self weight of support beam

$$W_{3SLS} = A_{sbu} \times \rho_{sb} = \mathbf{0.20 \text{ kN/m}}$$

Eccentricities

Distance to shear centre of main beam

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

Eccentricity of support beam masonry

$$e_{1mb} = (B_{mb} + b_{msb}) / 2 + c - e_{mb} = \mathbf{202 \text{ mm}}$$

Eccentricity of main beam masonry

$$e_{2mb} = (B_{mb} - b_{mmb}) / 2 - e_{mb} = \mathbf{-23 \text{ mm}}$$

Eccentricity of support beam

$$e_{3mb} = B_{mb} / 2 + c_{yysb} = \mathbf{102 \text{ mm}}$$

Torsional effects

Applied torque (ULS)

$$T_{qULS} = \text{abs}(W_{1ULS} \times e_{1mb} + W_{2ULS} \times e_{2mb} + W_{3ULS} \times e_{3mb}) = \mathbf{0.97 \text{ kNm/m}}$$

Total torque (ULS)

$$T_q = T_{qULS} \times L = \mathbf{3.61 \text{ kNm}}$$

Applied torque (SLS)

$$T_{qSLS} = \text{abs}(W_{1SLS} \times e_{1mb} + W_{2SLS} \times e_{2mb} + W_{3SLS} \times e_{3mb}) = \mathbf{0.69 \text{ kNm/m}}$$

Total torque (SLS)

$$T_{qu} = T_{qSLS} \times L = \mathbf{2.59 \text{ kNm}}$$

STEEL BEAM TORSION DESIGN

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

Tedds calculation version 2.0.02

Section details

Section type

UB 254x146x43

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

$$L = \mathbf{3725 \text{ mm}}$$

Length of segment for LT buckling

$$L_{LT} = \mathbf{3725 \text{ mm}}$$

Compression flanges laterally unrestrained

Partial torsional restraint against rotation about longitudinal axis provided by connection of bottom flange to supports

Effective length for LT buckling

$$L_{E,LT} = L_{LT} \times 1.2 + 2 \times D = \mathbf{4989 \text{ mm}}$$

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

$$F_{vy} = \mathbf{27.2 \text{ kN}}$$

Maximum bending moment

$$M_{LT} = M_x = \mathbf{25.35 \text{ kNm}}$$

Applied torque

$$T_q = \mathbf{3.61 \text{ kNm}}$$

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 fixed at each end. (as defined in SCI-P-057 section 2.1.6) - Appendix B case 6

Dist along the beam for first derivative of twist

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

Dist along the beam for second derivative of twist

$$z_2 = 0.2 \times L = \mathbf{745 \text{ mm}}$$

Dist along the beam for third derivative of twist

$$z_3 = L / 2 = \mathbf{1863 \text{ mm}}$$

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Second derivative of angle of twist

$$\phi''_1 = T_q / (2 \times G \times J \times a) \times [(1 + \cosh(L/a)) / \sinh(L/a) \times \cosh(z_1/a) - 2 \times a/L - \sinh(z_1/a)] \times 1 \text{ rads} = \mathbf{4.45 \times 10^{-2} \text{ rads/m}^2}$$

Third derivative of angle of twist

$$\phi'''_1 = T_q / (2 \times G \times J \times a^2) \times [(1 + \cosh(L/a)) / \sinh(L/a) \times \sinh(z_1/a) - \cosh(z_1/a)] \times 1 \text{ rads} = \mathbf{-8.52 \times 10^{-2} \text{ rads/m}^3}$$

First derivative of angle of twist

$$\phi'_2 = T_q / (2 \times G \times J) \times [(1 + \cosh(L/a)) / \sinh(L/a) \times \sinh(z_2/a) + (1 - 2 \times z_2/L) - \cosh(z_2/a)] \times 1 \text{ rads} = \mathbf{1.45 \times 10^{-2} \text{ rads/m}}$$

Angle of twist

$$\phi_3 = T_q \times a / (2 \times G \times J) \times [(1 + \cosh(L/a)) / \sinh(L/a) \times (\cosh(z_3/a) - 1) + z_3/a \times (1 - z_3/L) - \sinh(z_3/a)] \times 1 \text{ rads} = \mathbf{0.018 \text{ rads}}$$

Design parameters

Total angle of twist

$$\phi = \text{abs}(\phi_3) = \mathbf{0.018 \text{ rads}}$$

First derivative of ϕ

$$\phi' = \text{abs}(\phi'_2) = \mathbf{1.45 \times 10^{-2} \text{ rads/m}}$$

Second derivative of ϕ

$$\phi'' = \text{abs}(\phi''_1) = \mathbf{4.45 \times 10^{-2} \text{ rads/m}^2}$$

Third derivative of ϕ

$$\phi''' = \text{abs}(\phi'''_1) = \mathbf{8.52 \times 10^{-2} \text{ rads/m}^3}$$

Section classification

$$b/T = \mathbf{5.8}$$

$$d/t = \mathbf{30.4}$$

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

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

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force

$$F_{vy} = \mathbf{27.2 \text{ kN}}$$

Design shear resistance (Cl. 4.2.3)

$$P_{vy} = 0.6 \times p_y \times A_{vy} = \mathbf{398.1 \text{ kN}}$$

Pass - Shear

Moment capacity (x-axis)

Design bending moment

$$M_x = \mathbf{25.4 \text{ kNm}}$$

Moment capacity

$$M_{cxu} = p_y \times S_x = \mathbf{201.0 \text{ kNm}}$$

Moment capacity low shear (Cl. 4.2.5.1)

$$M_{cx} = \min(p_y \times S_x, 1.2 \times p_y \times Z_x) = \mathbf{201.0 \text{ kNm}}$$

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for lateral torsional buckling

$$L_{E_LT} = \mathbf{4989 \text{ mm}}$$

Slenderness ratio

$$\lambda = L_{E_LT} / r_y = \mathbf{142}$$

Buckling parameter

$$u = \mathbf{0.891}$$

Flange ratio

$$\eta = \mathbf{0.5}$$

Torsional index

$$x = \mathbf{21.2}$$

Slenderness factor

$$v = 1 / (1 + 0.05 \times (\lambda/x)^{0.25}) = \mathbf{0.74}$$

Ratio - cl 4.3.6.9

$$\beta_w = 1.0 = \mathbf{1.000}$$

Equivalent slenderness – cl 4.3.6.7

$$\lambda_{LT} = u \times v \times \lambda \times \sqrt{\beta_w} = \mathbf{94}$$

Limiting slenderness – Annex B2.2

$$\lambda_{L0} = 0.4 \times \sqrt{(\pi^2 \times E_{S5950} / p_y)} = \mathbf{30}$$

Euler stress

$$p_E = \pi^2 \times E_{S5950} / \lambda_{LT}^2 = \mathbf{228 \text{ N/mm}^2}$$

Perry factor

$$\eta_{LT} = \max(7.0 \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.447}$$

$$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{342806558.720}$$

Bending strength

$$p_b = p_E \times p_y / (\phi_{LT} + \sqrt{(\phi_{LT}^2 - p_E \times p_y)}) = \mathbf{152 \text{ N/mm}^2}$$

Buckling resistance moment

$$M_b = p_b \times S_x = \mathbf{86.0 \text{ kNm}}$$

Max moment governing buckling resistance

$$M_{LT} = \mathbf{25.4 \text{ kNm}}$$

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Equiv uniform moment factor for LTB

$$m_{LT} = 1.00$$

$$M_b / m_{LT} = 86.0 \text{ kNm}$$

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

$$L / a = 3.51$$

Angle of twist

$$\phi = 0.018 \text{ rads}$$

Second derivative of ϕ

$$\phi'' = 44.5 \times 10^{-3} \text{ rads/m}^2$$

Induced minor axis moment

$$M_{yt} = M_x \times \phi / 1 \text{ rad} = 0.44 \text{ kNm}$$

Normal stress at flange tip due to M_{yt}

$$\sigma_{byt} = M_{yt} / Z_y = 5 \text{ N/mm}^2$$

Normal stress at flange tip due to warping

$$\sigma_w = E_{S5950} \times W_{n0} \times \phi'' / 1 \text{ rad} = 83 \text{ N/mm}^2$$

Interaction index

$$i_b = M_x \times m_{LT} / M_b + (\sigma_{byt} + \sigma_w) / p_y \times (1 + 0.5 \times M_x \times m_{LT} / M_b) = 0.58$$

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

$$\sigma_{bx} = M_x / Z_x = 50 \text{ N/mm}^2$$

Combined stress - eqn 2.22

$$\sigma_{bx} + \sigma_{byt} + \sigma_w = 138 \text{ N/mm}^2$$

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

$$\tau_{tw} = \text{abs}(G \times t \times \phi' / 1 \text{ rad}) = 8 \text{ N/mm}^2$$

Max shear stresses due to torsion in flange

$$\tau_{tf} = \text{abs}(G \times T \times \phi' / 1 \text{ rad}) = 15 \text{ N/mm}^2$$

Max shear stresses due to warping in flange

$$\tau_{wf} = \text{abs}(-E_{S5950} \times S_{w1} \times \phi''' / 1 \text{ rad} / T) = 6 \text{ N/mm}^2$$

Amp shear stress torsion & warping in web

$$\tau_{vtw} = \tau_{tw} \times (1 + 0.5 \times M_x \times m_{LT} / M_b) = 9 \text{ N/mm}^2$$

Amp shear stress torsion & warping in flange

$$\tau_{vtf} = (\tau_{tf} + \tau_{wf}) \times (1 + 0.5 \times M_x \times m_{LT} / M_b) = 23 \text{ N/mm}^2$$

Combined shear stresses due to bending, torsion & warping:

Combined shear stresses in web

$$\tau_w = \tau_{bw} + \tau_{vtw} = 26 \text{ N/mm}^2$$

Combined shear stresses in flange

$$\tau_f = \tau_{bf} + \tau_{vtf} = 27 \text{ N/mm}^2$$

Shear strength

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

Pass - Combined shear stresses

Twist check

Total applied torque (unfactored)

$$T_{qu} = 2.59 \text{ kNm}$$

Maximum twist under sls loading

$$\phi_{sls} = \phi \times T_{qu} / T_q = 0.72 \text{ deg}$$

Twist limit

$$\phi_{lim} = 1.00 \text{ deg}$$

Pass - Twist

Deflection

Maximum y-axis deflection

$$\delta_{y_max} = 1.9 \text{ mm}$$

Deflection limit - cl. 2.5.2

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

Pass - Deflection within specified limit

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	Calcs for 200x100x8.0 RHS Post				Start page no./Revision 42	
	Calcs by LB	Calcs date 19/12/2019	Checked by LB	Checked date 28/01/2020	Approved by	Approved date

STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.07

Section details

Section type

RHS 200x100x8.0 (Tata Steel Celsius)

Steel grade

S275

From table 9: Design strength p_y

Thickness of element

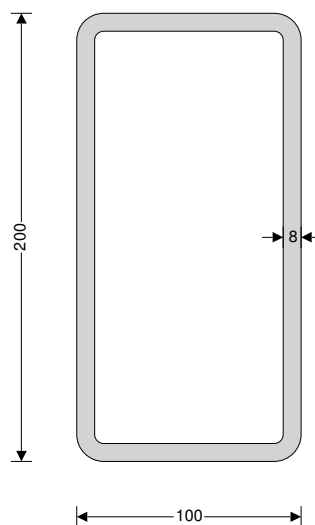
$t = 8.0$ mm

Design strength

$p_y = 275$ N/mm²

Modulus of elasticity

$E = 205000$ N/mm²



Lateral restraint

Distance between major axis restraints

$L_x = 2400$ mm

Distance between minor axis restraints

$L_y = 2400$ mm

Effective length factors

Effective length factor in major axis

$K_x = 1.00$

Effective length factor in minor axis

$K_y = 1.00$

Effective length factor for lateral-torsional buckling

$K_{LT} = 1.20 + 2 \times D$

Classification of cross sections - Section 3.5

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

Web - minor axis - Table 12

Depth of section

$d = B - 3 \times t = 76$ mm

Stress ratios

$r1 = \min(F_c / (2 \times d \times t \times p_{yw}), 1) = 0.449$

$r2 = F_c / (A \times p_{yw}) = 0.122$

$d / t = 9.5 \times \epsilon \leq \max(64 \times \epsilon / (1 + r1), 40 \times \epsilon)$ Class 1 plastic

Flange - minor axis - Table 12

Width of section

$b = D - 3 \times t = 176$ mm

$b / t = 22.0 \times \epsilon \leq 40 \times \epsilon$

Class 3 semi-compact

Section is class 3 semi-compact

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	Gethin House				LS378	
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	LB	19/12/2019	LB	28/01/2020		

Shear capacity - Section 4.2.3

Design shear force

$$F_{y,v} = 0 \text{ kN}$$

$$(D - 3 \times t) / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear capacity - Section 4.2.3

Design shear force

$$F_{x,v} = 10 \text{ kN}$$

Shear area

$$A_v = A_x = 1492 \text{ mm}^2$$

Design shear resistance

$$P_{x,v} = 0.6 \times p_y \times A_v = 246.1 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = 10 \text{ kNm}$$

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange

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

Limiting value for class 3 semi-compact flange

$$\beta_{3f} = 40 \times \epsilon = 40$$

Limiting value for class 2 compact web

$$\beta_{2w} = \max(80 \times \epsilon / (1 + r_1), 40 \times \epsilon) = 55.227$$

Limiting value for class 3 semi-compact web

$$\beta_{3w} = \max(120 \times \epsilon / (1 + 2 \times r_2), 40 \times \epsilon) = 96.482$$

Effective plastic modulus - cl.3.5.6.3

$$S_{eff} = \min(Z_{yy} + (S_{yy} - Z_{yy}) \times \min([\beta_{3w} / (b / t) - 1] / (\beta_{3w} / \beta_{2w} - 1), [(\beta_{3f} / (d / t) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S_{yy}) = 171784 \text{ mm}^3$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z_{yy}) = 47.2 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force

$$F_c = 150 \text{ kN}$$

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

Effective length for buckling

$$L_{Ex} = L_x \times K_x = 2400 \text{ mm}$$

Slenderness ratio - cl.4.7.2

$$\lambda_x = L_{Ex} / r_{xx} = 33.972$$

Compressive strength - Section 4.7.5

Limiting slenderness

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

Strut curve - Table 23

$$a$$

Robertson constant

$$\alpha_x = 2.0$$

Perry factor

$$\eta_x = \alpha_x \times (\lambda_x - \lambda_0) / 1000 = 0.034$$

Euler stress

$$p_{Ex} = \pi^2 \times E / \lambda_x^2 = 1753.1 \text{ N/mm}^2$$

$$\phi_x = (p_y + (\eta_x + 1) \times p_{Ex}) / 2 = 1043.5 \text{ N/mm}^2$$

Compressive strength - Annex C.1

$$p_{cx} = p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5}) = 264.5 \text{ N/mm}^2$$

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4

$$P_{cx} = A \times p_{cx} = 1183.8 \text{ kN}$$

PASS - Compression resistance exceeds design compression force

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

Effective length for buckling

$$L_{Ey} = L_y \times K_y = 2400 \text{ mm}$$

Slenderness ratio - cl.4.7.2

$$\lambda_y = L_{Ey} / r_{yy} = 59.061$$

Compressive strength - Section 4.7.5

Limiting slenderness

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

Strut curve - Table 23

$$a$$

Robertson constant

$$\alpha_y = 2.0$$

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Perry factor

$$\eta_y = \alpha_y \times (\lambda_y - \lambda_0) / 1000 = \mathbf{0.084}$$

Euler stress

$$p_{Ey} = \pi^2 \times E / \lambda_y^2 = \mathbf{580 \text{ N/mm}^2}$$

$$\phi_y = (p_y + (\eta_y + 1) \times p_{Ey}) / 2 = \mathbf{451.8 \text{ N/mm}^2}$$

Compressive strength - Annex C.1

$$p_{cy} = p_{Ey} \times p_y / (\phi_y + (\phi_y^2 - p_{Ey} \times p_y)^{0.5}) = \mathbf{240.6 \text{ N/mm}^2}$$

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4

$$P_{cy} = A \times p_{cy} = \mathbf{1076.6 \text{ kN}}$$

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comb.compression & bending check - cl.4.8.3.2

$$F_c / (A \times p_y) + M / M_c = \mathbf{0.334}$$

PASS - Combined bending and compression check is satisfied

Member buckling resistance - Section 4.8.3.3

Max major axis moment governing M_b

$$M_{LT} = M_x = \mathbf{0.00 \text{ kNm}}$$

Equivalent uniform moment factor for major axis flexural buckling

$$m_x = \mathbf{1.000}$$

$$m_y = \mathbf{1.000}$$

Buckling resistance checks - cl.4.8.3.3.3

$$F_c / P_{cy} + m_y \times M / M_c \times (1 + 0.5 \times F_c / P_{cy}) = \mathbf{0.366}$$

$$F_c / P_{cx} + 0.5 \times m_{yx} \times M_y / M_{cy} = \mathbf{0.233}$$

PASS - Member buckling resistance checks are satisfied

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STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.07

Section details

Section type

SHS 100x100x6.3 (Tata Steel Celsius)

Steel grade

S275

From table 9: Design strength p_y

Thickness of element

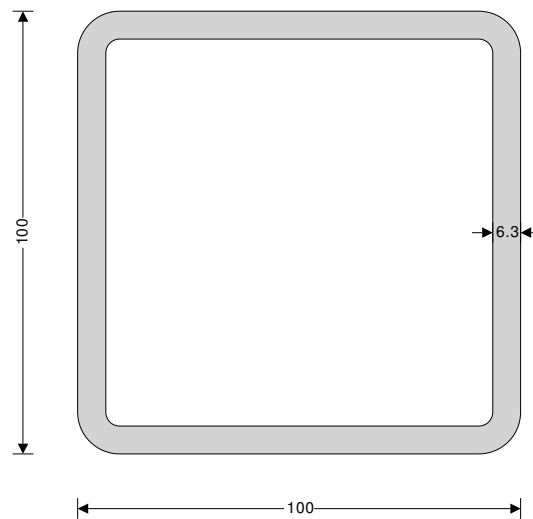
$t = 6.3 \text{ mm}$

Design strength

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

Modulus of elasticity

$E = 205000 \text{ N/mm}^2$



Lateral restraint

Distance between major axis restraints

$L_x = 2400 \text{ mm}$

Distance between minor axis restraints

$L_y = 2400 \text{ mm}$

Effective length factors

Effective length factor in major axis

$K_x = 1.00$

Effective length factor in minor axis

$K_y = 1.00$

Effective length factor for lateral-torsional buckling

$K_{LT} = 1.20 + 2 \times D$

Classification of cross sections - Section 3.5

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

Web - minor axis - Table 12

Depth of section

$$d = B - 3 \times t = 81.1 \text{ mm}$$

Stress ratios

$$r1 = \min(F_c / (2 \times d \times t \times p_{yw}), 1) = 0.214$$

$$r2 = F_c / (A \times p_{yw}) = 0.094$$

$$d / t = 12.9 \times \varepsilon \leq \max(64 \times \varepsilon / (1 + r1), 40 \times \varepsilon) \quad \text{Class 1 plastic}$$

Flange - minor axis - Table 12

Width of section

$$b = D - 3 \times t = 81.1 \text{ mm}$$

$$b / t = 12.9 \times \varepsilon \leq 40 \times \varepsilon$$

Class 3 semi-compact

Section is class 3 semi-compact

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
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Shear capacity - Section 4.2.3

Design shear force

$$F_{y,v} = 0 \text{ kN}$$

$$(D - 3 \times t) / t < 70 \times \epsilon$$

Web does not need to be checked for shear buckling

Shear capacity - Section 4.2.3

Design shear force

$$F_{x,v} = 6 \text{ kN}$$

Shear area

$$A_v = A_x = 1159 \text{ mm}^2$$

Design shear resistance

$$P_{x,v} = 0.6 \times p_y \times A_v = 191.3 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment

$$M = 6 \text{ kNm}$$

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange

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

Limiting value for class 3 semi-compact flange

$$\beta_{3f} = 40 \times \epsilon = 40$$

Limiting value for class 2 compact web

$$\beta_{2w} = \max(80 \times \epsilon / (1 + r_1), 40 \times \epsilon) = 65.924$$

Limiting value for class 3 semi-compact web

$$\beta_{3w} = \max(120 \times \epsilon / (1 + 2 \times r_2), 40 \times \epsilon) = 100.993$$

Effective plastic modulus - cl.3.5.6.3

$$S_{eff} = \min(Z + (S - Z) \times \min([\beta_{3w} / (d / t) - 1] / (\beta_{3w} / \beta_{2w} - 1), [(\beta_{3f} / (b / t) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S) = 80863 \text{ mm}^3$$

Moment capacity low shear - cl.4.2.5.2

$$M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z) = 22.1 \text{ kNm}$$

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force

$$F_c = 60 \text{ kN}$$

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

Effective length for buckling

$$L_{Ex} = L_x \times K_x = 2400 \text{ mm}$$

Slenderness ratio - cl.4.7.2

$$\lambda_x = L_{Ex} / r_{xx} = 63.086$$

Compressive strength - Section 4.7.5

Limiting slenderness

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

Strut curve - Table 23

$$a$$

Robertson constant

$$\alpha_x = 2.0$$

Perry factor

$$\eta_x = \alpha_x \times (\lambda_x - \lambda_0) / 1000 = 0.092$$

Euler stress

$$p_{Ex} = \pi^2 \times E / \lambda_x^2 = 508.4 \text{ N/mm}^2$$

$$\phi_x = (p_y + (\eta_x + 1) \times p_{Ex}) / 2 = 415 \text{ N/mm}^2$$

Compressive strength - Annex C.1

$$p_{cx} = p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5}) = 234.9 \text{ N/mm}^2$$

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4

$$P_{cx} = A \times p_{cx} = 544.6 \text{ kN}$$

PASS - Compression resistance exceeds design compression force

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

Effective length for buckling

$$L_{Ey} = L_y \times K_y = 2400 \text{ mm}$$

Slenderness ratio - cl.4.7.2

$$\lambda_y = L_{Ey} / r_{yy} = 63.086$$

Compressive strength - Section 4.7.5

Limiting slenderness

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

Strut curve - Table 23

$$a$$

Robertson constant

$$\alpha_y = 2.0$$

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Perry factor

$$\eta_y = \alpha_y \times (\lambda_y - \lambda_0) / 1000 = \mathbf{0.092}$$

Euler stress

$$p_{Ey} = \pi^2 \times E / \lambda_y^2 = \mathbf{508.4 \text{ N/mm}^2}$$

$$\phi_y = (p_y + (\eta_y + 1) \times p_{Ey}) / 2 = \mathbf{415 \text{ N/mm}^2}$$

Compressive strength - Annex C.1

$$p_{cy} = p_{Ey} \times p_y / (\phi_y + (\phi_y^2 - p_{Ey} \times p_y)^{0.5}) = \mathbf{234.9 \text{ N/mm}^2}$$

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4

$$P_{cy} = A \times p_{cy} = \mathbf{544.6 \text{ kN}}$$

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comb.compression & bending check - cl.4.8.3.2

$$F_c / (A \times p_y) + M / M_c = \mathbf{0.365}$$

PASS - Combined bending and compression check is satisfied

Member buckling resistance - Section 4.8.3.3

Max major axis moment governing M_b

$$M_{LT} = M_x = \mathbf{0.00 \text{ kNm}}$$

Equivalent uniform moment factor for major axis flexural buckling

$$m_x = \mathbf{1.000}$$

$$m_y = \mathbf{1.000}$$

Buckling resistance checks - cl.4.8.3.3.3

$$F_c / P_{cy} + m_y \times M / M_c \times (1 + 0.5 \times F_c / P_{cy}) = \mathbf{0.396}$$

$$F_c / P_{cx} + 0.5 \times m_{yx} \times M_y / M_{cy} = \mathbf{0.246}$$

PASS - Member buckling resistance checks are satisfied

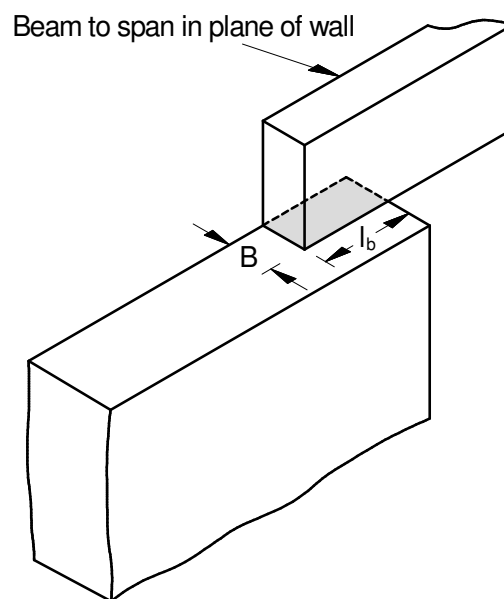
 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
	Gethin House				LS378	
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	Masonry Bearing Check				48	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	LB	19/12/2019	LB	28/01/2020		

MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.06

Masonry details

Masonry type	Aggregate concrete blocks (25% or less formed voids)
Compressive strength of unit	$p_{unit} = 7.3 \text{ N/mm}^2$
Mortar designation	iii
Least horizontal dimension of masonry units	$l_{unit} = 100 \text{ mm}$
Height of masonry units	$h_{unit} = 215 \text{ mm}$
Category of masonry units	Category II
Category of construction control	Normal
Partial safety factor for material strength	$\gamma_m = 3.5$
Thickness of load bearing leaf	$t = 140 \text{ mm}$
Effective thickness of masonry wall	$t_{ef} = 140 \text{ mm}$
Height of masonry wall	$h = 2400 \text{ mm}$
Effective height of masonry wall	$h_{ef} = 2400 \text{ mm}$



Bearing details

Beam spanning in plane of wall	
Width of bearing	$B = 100 \text{ mm}$
Length of bearing	$l_b = 250 \text{ mm}$

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

Mortar designation	Mortar = "iii"
Block compressive strength	$p_{unit} = 7.3 \text{ N/mm}^2$
Characteristic compressive strength (Table 2c)	$f_{kc} = 3.20 \text{ N/mm}^2$
Characteristic compressive strength (Table 2d)	$f_{kd} = 6.40 \text{ N/mm}^2$
Height of solid block	$h_{unit} = 215.0 \text{ mm}$
Least horizontal dimension	$l_{unit} = 100.0 \text{ mm}$
Block ratio	$ratio = h_{unit} / l_{unit} = 2.2$

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	LB	19/12/2019	LB	28/01/2020		

Ratio between 0.6 and 4.5 - OK

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

Loading details

Characteristic concentrated dead load $G_k = 20 \text{ kN}$

Characteristic concentrated imposed load $Q_k = 23 \text{ kN}$

Design concentrated load $F = (G_k \times 1.4) + (Q_k \times 1.6) = 63.3 \text{ kN}$

Characteristic distributed dead load $g_k = 0.0 \text{ kN/m}$

Characteristic distributed imposed load $q_k = 0.0 \text{ kN/m}$

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

Masonry bearing type

Bearing type **Type 2**

Bearing safety factor $\gamma_{\text{bear}} = 1.50$

Check design bearing without a spreader

Design bearing stress $f_{ca} = F / (B \times l_b) + f / t = 2.532 \text{ N/mm}^2$

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

PASS - Allowable bearing stress exceeds design bearing stress

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

Slenderness ratio $h_{\text{ef}} / t_{\text{ef}} = 17.14$

Eccentricity at top of wall $e_x = 0.0 \text{ mm}$

From BS5628:1 Table 7

Capacity reduction factor $\beta = 0.86$

Length of bearing distributed at $0.4 \times h$ $l_d = 1210 \text{ mm}$

Maximum bearing stress $f_{ca} = F / (l_d \times t) + f / t = 0.374 \text{ N/mm}^2$

Allowable bearing stress $f_{cp} = \beta \times f_k / \gamma_m = 1.579 \text{ N/mm}^2$

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

RAFT FOUNDATION CALCULATIONS

TO BE READ IN CONJUNCTION WITH STRUCTURAL REFERENCES AND ARCHITECTS DRAWINGS

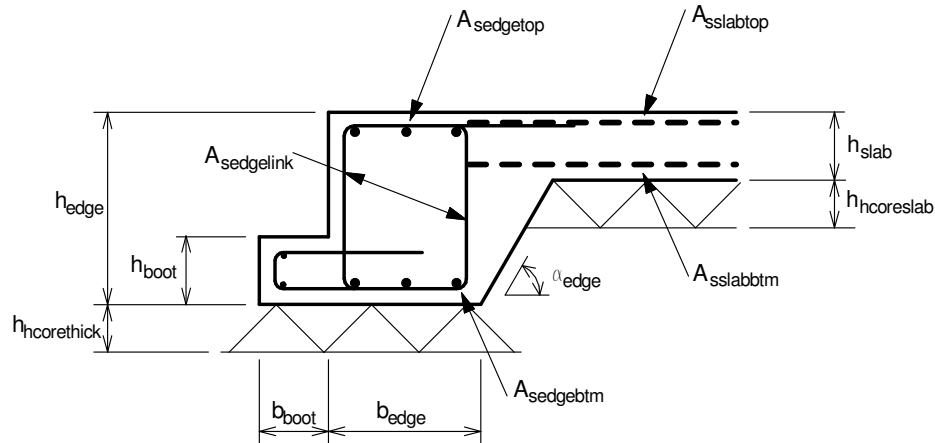
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e:lewis@lbstructural.co.uk

LB STRUCTURAL DESIGN
21 HEOL STRADLING
COITY, BRIDGEND
CF35 6AN

 Tekla Tedds LB Structural Design 21 Heol Stradling Coity Bridgend, CF35 6AN	Project				Job no.	
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	Raft Foundation Design				51	
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	LB	19/12/2019	LB	28/01/2020		

RAFT FOUNDATION DESIGN (BS8110 : PART 1 : 1997)

Tedds calculation version 1.0.12



Soil and raft definition

Soil definition

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

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

One type only

Firm to loose

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

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

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

$$\phi_{\text{depbasic}} = 2000 \text{ mm}$$

$$\phi_{\text{dep slab}} = \phi_{\text{depbasic}} - h_{\text{coreslab}} = 1850 \text{ mm}$$

$$\phi_{\text{dep thick}} = \phi_{\text{depbasic}} - h_{\text{corethick}} = 1850 \text{ mm}$$

Raft slab definition

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

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

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

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

$$\nu = 0.2$$

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

$$\gamma_s = 1.15$$

Minimum mesh required in top for shrinkage

A142

Actual mesh provided in top

$$\mathbf{A393} \text{ (} A_{\text{sslabtop}} = 393 \text{ mm}^2/\text{m} \text{)}$$

Mesh provided in bottom

$$\mathbf{A393} \text{ (} A_{\text{sslabbtm}} = 393 \text{ mm}^2/\text{m} \text{)}$$

Top mesh bar diameter

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

Bottom mesh bar diameter

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

Cover to top reinforcement

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

Cover to bottom reinforcement

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

Average effective depth of top reinforcement

$$d_{\text{t slabav}} = h_{\text{slab}} - C_{\text{top}} - \phi_{\text{slabtop}} = 105 \text{ mm}$$

Average effective depth of bottom reinforcement

$$d_{\text{b slabav}} = h_{\text{slab}} - C_{\text{btm}} - \phi_{\text{slabbtm}} = 105 \text{ mm}$$

Overall average effective depth

$$d_{\text{slabav}} = (d_{\text{t slabav}} + d_{\text{b slabav}})/2 = 105 \text{ mm}$$

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Minimum effective depth of top reinforcement

$$d_{slabmin} = d_{slabav} - \phi_{slabtop}/2 = 100 \text{ mm}$$

Minimum effective depth of bottom reinforcement

$$d_{bslabmin} = d_{bslabav} - \phi_{slabbtm}/2 = 100 \text{ mm}$$

Edge beam definition

Overall depth

$$h_{edge} = 675 \text{ mm}$$

Width

$$b_{edge} = 450 \text{ mm}$$

Depth of boot

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

Width of boot

$$b_{boot} = 175 \text{ mm}$$

Angle of chamfer to horizontal

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

Strength of main bar reinforcement

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

Strength of link reinforcement

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

Reinforcement provided in top

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

Reinforcement provided in bottom

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

Link reinforcement provided

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

Bottom cover to links

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

Effective depth of top reinforcement

$$d_{edgetop} = h_{edge} - C_{top} - \phi_{slabtop} - \phi_{edgelink} - \phi_{edgetop}/2 = 612 \text{ mm}$$

Effective depth of bottom reinforcement

$$d_{edge\text{btm}} = h_{edge} - C_{beam} - \phi_{edgelink} - \phi_{edge\text{btm}}/2 = 622 \text{ mm}$$

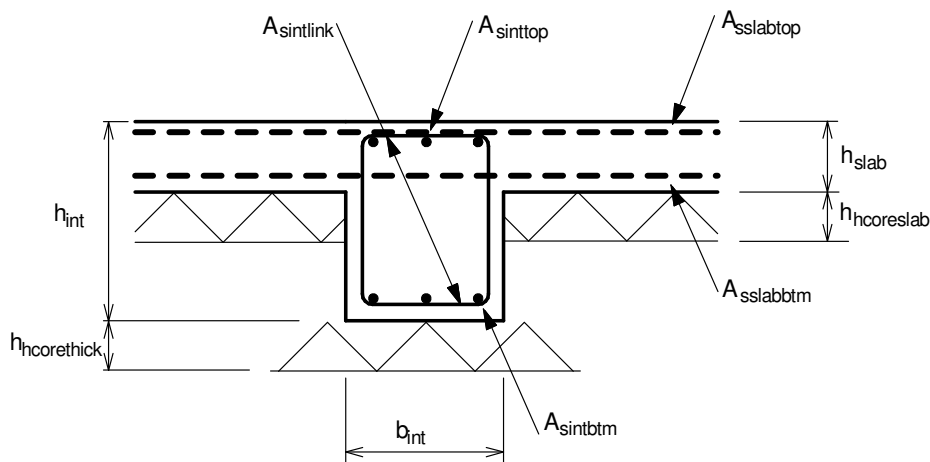
Boot main reinforcement

$$\text{H10 bars at 250 ctrs } (A_{sboot} = 314 \text{ mm}^2/\text{m})$$

Effective depth of boot reinforcement

$$d_{boot} = h_{boot} - C_{beam} - \phi_{boot}/2 = 185 \text{ mm}$$

Internal beam definition



Overall depth

$$h_{int} = 450 \text{ mm}$$

Width

$$b_{int} = 450 \text{ mm}$$

Strength of main bar reinforcement

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

Strength of link reinforcement

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

Reinforcement provided in top

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

Reinforcement provided in bottom

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

Link reinforcement provided

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

Effective depth of top reinforcement

$$d_{int\text{top}} = h_{int} - C_{top} - 2 \times \phi_{slabtop} - \phi_{int\text{top}}/2 = 387 \text{ mm}$$

Effective depth of bottom reinforcement

$$d_{int\text{btm}} = h_{int} - C_{beam} - \phi_{intlink} - \phi_{int\text{btm}}/2 = 397 \text{ mm}$$

Internal slab design checks

Basic loading

Slab self weight

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

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Hardcore

$$W_{hcoreslab} = \gamma_{hcore} \times h_{hcoreslab} = 2.7 \text{ kN/m}^2$$

Applied loading

Uniformly distributed dead load

$$W_{Dudl} = 5.0 \text{ kN/m}^2$$

Uniformly distributed live load

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

Internal slab bearing pressure check

Total uniform load at formation level

$$W_{udl} = W_{slab} + W_{hcoreslab} + W_{Dudl} + W_{Ludl} = 12.8 \text{ kN/m}^2$$

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

Internal slab bending and shear check

Applied bending moments

Span of slab

$$l_{slab} = \phi_{depslab} + d_{slabav} = 1955 \text{ mm}$$

Ultimate self weight udl

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

Self weight moment at centre

$$M_{csw} = W_{swult} \times l_{slab}^2 \times (1 + \nu) / 64 = 0.4 \text{ kNm/m}$$

Self weight moment at edge

$$M_{esw} = W_{swult} \times l_{slab}^2 / 32 = 0.6 \text{ kNm/m}$$

Self weight shear force at edge

$$V_{sw} = W_{swult} \times l_{slab} / 4 = 2.5 \text{ kN/m}$$

Moments due to applied uniformly distributed loads

Ultimate applied udl

$$W_{udult} = 1.4 \times W_{Dudl} + 1.6 \times W_{Ludl} = 9.4 \text{ kN/m}^2$$

Moment at centre

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

Moment at edge

$$M_{eudl} = W_{udult} \times l_{slab}^2 / 32 = 1.1 \text{ kNm/m}$$

Shear force at edge

$$V_{udl} = W_{udult} \times l_{slab} / 4 = 4.6 \text{ kN/m}$$

Resultant moments and shears

Total moment at edge

$$M_{\Sigma e} = 1.7 \text{ kNm/m}$$

Total moment at centre

$$M_{\Sigma c} = 1.0 \text{ kNm/m}$$

Total shear force

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

Reinforcement required in top

K factor

$$K_{slabtop} = M_{\Sigma e} / (f_{cu} \times d_{slabav}^2) = 0.004$$

Lever arm

$$Z_{slabtop} = d_{slabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtop}/0.9)}) = 99.7 \text{ mm}$$

Area of steel required for bending

$$A_{sslabtopbend} = M_{\Sigma e} / ((1.0/\gamma_s) \times f_{yslab} \times Z_{slabtop}) = 40 \text{ mm}^2/\text{m}$$

Minimum area of steel required

$$A_{sslabmin} = 0.0013 \times h_{slab} = 195 \text{ mm}^2/\text{m}$$

Area of steel required

$$A_{sslabtopreq} = \max(A_{sslabtopbend}, A_{sslabmin}) = 195 \text{ mm}^2/\text{m}$$

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

Reinforcement required in bottom

K factor

$$K_{slabbtm} = M_{\Sigma c} / (f_{cu} \times d_{slabav}^2) = 0.003$$

Lever arm

$$Z_{slabbtm} = d_{slabav} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbtm}/0.9)}) = 99.7 \text{ mm}$$

Area of steel required for bending

$$A_{sslabbtmbend} = M_{\Sigma c} / ((1.0/\gamma_s) \times f_{yslab} \times Z_{slabbtm}) = 24 \text{ mm}^2/\text{m}$$

Area of steel required

$$A_{sslabbtmreq} = \max(A_{sslabbtmbend}, A_{sslabmin}) = 195 \text{ mm}^2/\text{m}$$

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

Shear check

Applied shear stress

$$v = V_{\Sigma} / d_{slabmin} = 0.071 \text{ N/mm}^2$$

Tension steel ratio

$$\rho = 100 \times A_{sslabtop} / d_{slabmin} = 0.393$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$v_c = 0.732 \text{ N/mm}^2$$

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

Internal slab deflection check

Basic allowable span to depth ratio

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

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Moment factor

$$M_{factor} = M_{\Sigma c} / d_{bslabav}^2 = \mathbf{0.094 \text{ N/mm}^2}$$

Steel service stress

$$f_s = 2/3 \times f_{yslab} \times A_{sslabbtmbend} / A_{sslabbtm} = \mathbf{20.238 \text{ N/mm}^2}$$

Modification factor

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

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

Modified allowable span to depth ratio

$$Ratio_{allow} = Ratio_{basic} \times MF_{slab} = \mathbf{52.000}$$

Actual span to depth ratio

$$Ratio_{actual} = l_{slab} / d_{bslabav} = \mathbf{18.619}$$

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

Edge beam design checks

Basic loading

Hardcore

$$W_{hcorethick} = \gamma_{hcore} \times h_{hcorethick} = \mathbf{2.7 \text{ kN/m}^2}$$

Edge beam

Rectangular beam element

$$W_{beam} = 24 \text{ kN/m}^3 \times h_{edge} \times b_{edge} = \mathbf{7.3 \text{ kN/m}}$$

Boot element

$$W_{boot} = 24 \text{ kN/m}^3 \times h_{boot} \times b_{boot} = \mathbf{0.9 \text{ kN/m}}$$

Chamfer element

$$W_{chamfer} = 24 \text{ kN/m}^3 \times (h_{edge} - h_{slab})^2 / (2 \times \tan(\alpha_{edge})) = \mathbf{1.9 \text{ kN/m}}$$

Slab element

$$W_{slabelmt} = 24 \text{ kN/m}^3 \times h_{slab} \times (h_{edge} - h_{slab}) / \tan(\alpha_{edge}) = \mathbf{1.1 \text{ kN/m}}$$

Edge beam self weight

$$W_{edge} = W_{beam} + W_{boot} + W_{chamfer} + W_{slabelmt} = \mathbf{11.2 \text{ kN/m}}$$

Edge load number 1

Load type

Longitudinal line load

Dead load

$$W_{Dedge1} = \mathbf{15.0 \text{ kN/m}}$$

Live load

$$W_{Ledge1} = \mathbf{3.0 \text{ kN/m}}$$

Ultimate load

$$W_{ultedge1} = 1.4 \times W_{Dedge1} + 1.6 \times W_{Ledge1} = \mathbf{25.8 \text{ kN/m}}$$

Longitudinal line load width

$$b_{edge1} = \mathbf{100 \text{ mm}}$$

Centroid of load from outside face of raft

$$X_{edge1} = \mathbf{270 \text{ mm}}$$

Edge load number 2

Load type

Longitudinal line load

Dead load

$$W_{Dedge2} = \mathbf{9.6 \text{ kN/m}}$$

Live load

$$W_{Ledge2} = \mathbf{0.0 \text{ kN/m}}$$

Ultimate load

$$W_{ultedge2} = 1.4 \times W_{Dedge2} + 1.6 \times W_{Ledge2} = \mathbf{13.4 \text{ kN/m}}$$

Longitudinal line load width

$$b_{edge2} = \mathbf{100 \text{ mm}}$$

Centroid of load from outside face of raft

$$X_{edge2} = \mathbf{75 \text{ mm}}$$

Edge beam bearing pressure check

Effective bearing width of edge beam

$$b_{bearing} = b_{edge} + b_{boot} + (h_{edge} - h_{slab}) / \tan(\alpha_{edge}) = \mathbf{928 \text{ mm}}$$

Total uniform load at formation level

$$W_{udledge} = W_{Dudl} + W_{Ludl} + W_{edge} / b_{bearing} + W_{hcorethick} = \mathbf{21.3 \text{ kN/m}^2}$$

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

Load x distance for edge load 1

$$Moment_1 = W_{ultedge1} \times X_{edge1} = \mathbf{7.0 \text{ kN}}$$

Load x distance for edge load 2

$$Moment_2 = W_{ultedge2} \times X_{edge2} = \mathbf{1.0 \text{ kN}}$$

Sum of ultimate longitud'l and equivalent line loads $\Sigma UDL = \mathbf{39.2 \text{ kN/m}}$

Sum of load x distances

$$\Sigma Moment = \mathbf{8.0 \text{ kN}}$$

Centroid of loads

$$X_{bar} = \Sigma Moment / \Sigma UDL = \mathbf{203 \text{ mm}}$$

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

Sum of unfactored longitud'l and eff'tive line loads $\Sigma UDLsls = \mathbf{27.6 \text{ kN/m}}$

Allowable bearing width

$$b_{allow} = 2 \times X_{bar} + 2 \times h_{hcoreslab} \times \tan(30) = \mathbf{580 \text{ mm}}$$

Bearing pressure due to line/point loads

$$q_{linepoint} = \Sigma UDLsls / b_{allow} = \mathbf{47.6 \text{ kN/m}^2}$$

Total applied bearing pressure

$$q_{edge} = q_{linepoint} + W_{udledge} = \mathbf{68.9 \text{ kN/m}^2}$$

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

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

Bearing width required	$b_{req} = \Sigma UDLs / (q_{allow} - w_{udledge}) = 962 \text{ mm}$
Effective bearing width at u/s of slab	$b_{reqeff} = b_{req} - 2 \times h_{ncore slab} \times \tan(30) = 789 \text{ mm}$
Load/reaction eccentricity	$e = b_{reqeff} / 2 - x_{bar} = 191 \text{ mm}$
Ultimate moment to be resisted by slab	$M_{ecc} = \Sigma UDL \times e = 7.5 \text{ kNm/m}$
From slab bending check	
Moment due to depression under slab (hogging)	$M_{\Sigma e} = 1.7 \text{ kNm/m}$
Total moment to be resisted by slab top steel	$M_{slabtop} = M_{ecc} + M_{\Sigma e} = 9.2 \text{ kNm/m}$
K factor	$K_{slab} = M_{slabtop} / (f_{cu} \times d_{slabmin}^2) = 0.026$
Lever arm	$Z_{slab} = d_{slabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slab}/0.9)}) = 95 \text{ mm}$
Area of steel required	$A_{sslabreq} = M_{slabtop} / ((1.0/\gamma_s) \times f_y \times Z_{slab}) = 223 \text{ mm}^2/\text{m}$

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

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

Library item - B pressure with moment transfer

Edge beam bending check

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

Span of edge beam	$l_{edge} = \phi_{depth} + d_{edgetop} = 2462 \text{ mm}$
Ultimate self weight udl	$w_{edgeult} = 1.4 \times w_{edge} = 15.7 \text{ kN/m}$
Ultimate slab udl (approx)	$w_{edgeslab} = \max(0 \text{ kN/m}, 1.4 \times w_{slab} \times ((\phi_{depth}/2 \times 3/4) - (b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}))))$
	$w_{edgeslab} = 0.0 \text{ kN/m}$
Self weight and slab bending moment	$M_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge}^2 / \beta_{udl} = 9.5 \text{ kNm}$
Self weight shear force	$V_{edgesw} = (w_{edgeult} + w_{edgeslab}) \times l_{edge} / 2 = 19.4 \text{ kN}$

Moments due to applied uniformly distributed loads

Ultimate udl (approx)	$w_{edgeudl} = w_{udlult} \times \phi_{depth} / 2 \times 3/4 = 6.5 \text{ kN/m}$
Bending moment	$M_{edgeudl} = w_{edgeudl} \times l_{edge}^2 / \beta_{udl} = 4.0 \text{ kNm}$
Shear force	$V_{edgeudl} = w_{edgeudl} \times l_{edge} / 2 = 8.0 \text{ kN}$

Moment and shear due to load number 1

Bending moment	$M_{edge1} = w_{ultedge1} \times l_{edge}^2 / \beta_{udl} = 15.6 \text{ kNm}$
Shear force	$V_{edge1} = w_{ultedge1} \times l_{edge} / 2 = 31.8 \text{ kN}$

Moment and shear due to load number 2

Bending moment	$M_{edge2} = w_{ultedge2} \times l_{edge}^2 / \beta_{udl} = 8.1 \text{ kNm}$
Shear force	$V_{edge2} = w_{ultedge2} \times l_{edge} / 2 = 16.5 \text{ kN}$

Resultant moments and shears

Total moment (hogging and sagging)	$M_{\Sigma edge} = 37.3 \text{ kNm}$
Maximum shear force	$V_{\Sigma edge} = 75.7 \text{ kN}$

Reinforcement required in top

Width of section in compression zone	$b_{edgetop} = b_{edge} + b_{boot} = 625 \text{ mm}$
Average web width	$b_w = b_{edge} + (h_{edge} / \tan(\alpha_{edge})) / 2 = 645 \text{ mm}$
K factor	$K_{edgetop} = M_{\Sigma edge} / (f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = 0.005$
Lever arm	$Z_{edgetop} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgetop}/0.9)}) = 581 \text{ mm}$
Area of steel required for bending	$A_{sedgetopbend} = M_{\Sigma edge} / ((1.0/\gamma_s) \times f_y \times Z_{edgetop}) = 147 \text{ mm}^2$
Minimum area of steel required	$A_{sedgetopmin} = 0.0013 \times 1.0 \times b_w \times h_{edge} = 566 \text{ mm}^2$
Area of steel required	$A_{sedgetopreq} = \max(A_{sedgetopbend}, A_{sedgetopmin}) = 566 \text{ mm}^2$

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

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Reinforcement required in bottom

Width of section in compression zone

$$b_{edgebtm} = b_{edge} + (h_{edge} - h_{slab})/\tan(\alpha_{edge}) + 0.1 \times l_{edge} = \mathbf{999 \text{ mm}}$$

K factor

$$K_{edgebtm} = M_{\Sigma edge}/(f_{cu} \times b_{edgebtm} \times d_{edgebtm}^2) = \mathbf{0.003}$$

Lever arm

$$Z_{edgebtm} = d_{edgebtm} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{edgebtm}/0.9)}) = \mathbf{591 \text{ mm}}$$

Area of steel required for bending

$$A_{sedgebtmbend} = M_{\Sigma edge}/((1.0/\gamma_s) \times f_y \times Z_{edgebtm}) = \mathbf{145 \text{ mm}^2}$$

Minimum area of steel required

$$A_{sedgebtmmin} = 0.0013 \times 1.0 \times b_w \times h_{edge} = \mathbf{566 \text{ mm}^2}$$

Area of steel required

$$A_{sedgebtmreq} = \max(A_{sedgebtmbend}, A_{sedgebtmmin}) = \mathbf{566 \text{ mm}^2}$$

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

Edge beam shear check

Applied shear stress

$$v_{edge} = V_{\Sigma edge}/(b_w \times d_{edgetop}) = \mathbf{0.192 \text{ N/mm}^2}$$

Tension steel ratio

$$\rho_{edge} = 100 \times A_{sedgetop}/(b_w \times d_{edgetop}) = \mathbf{0.153}$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$v_{cedge} = \mathbf{0.378 \text{ N/mm}^2}$$

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

Link area to spacing ratio required

$$A_{sv_upon_svreqedge} = 0.4 \text{ N/mm}^2 \times b_w/((1.0/\gamma_s) \times f_{ys}) = \mathbf{0.593 \text{ mm}}$$

Link area to spacing ratio provided

$$A_{sv_upon_svprovedge} = N_{edgeline} \times \pi \times \phi_{edgeline}^2 / (4 \times s_{vedge}) = \mathbf{0.628 \text{ mm}}$$

PASS - $A_{sv_upon_svreqedge} \leq A_{sv_upon_svprovedge}$ - Shear reinforcement provided in edge beams is adequate

Boot design check

Effective cantilever span

$$l_{boot} = b_{boot} + d_{boot}/2 = \mathbf{268 \text{ mm}}$$

Approximate ultimate bearing pressure

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

Cantilever moment

$$M_{boot} = q_{ult} \times l_{boot}^2/2 = \mathbf{2.8 \text{ kNm/m}}$$

Shear force

$$V_{boot} = q_{ult} \times l_{boot} = \mathbf{20.7 \text{ kN/m}}$$

K factor

$$K_{boot} = M_{boot}/(f_{cu} \times d_{boot}^2) = \mathbf{0.002}$$

Lever arm

$$Z_{boot} = d_{boot} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{boot}/0.9)}) = \mathbf{176 \text{ mm}}$$

Area of reinforcement required

$$A_{sbootreq} = M_{boot}/((1.0/\gamma_s) \times f_y \times Z_{boot}) = \mathbf{36 \text{ mm}^2/\text{m}}$$

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

Applied shear stress

$$v_{boot} = V_{boot}/d_{boot} = \mathbf{0.112 \text{ N/mm}^2}$$

Tension steel ratio

$$\rho_{boot} = 100 \times A_{sboot}/d_{boot} = \mathbf{0.170}$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$v_{cboot} = \mathbf{0.475 \text{ N/mm}^2}$$

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

Corner design checks

Basic loading

Corner load number 1

Load type

Line load in x direction

Dead load

$$w_{Dcorner1} = \mathbf{15.0 \text{ kN/m}}$$

Live load

$$w_{Lcorner1} = \mathbf{3.0 \text{ kN/m}}$$

Ultimate load

$$w_{ultcorner1} = 1.4 \times w_{Dcorner1} + 1.6 \times w_{Lcorner1} = \mathbf{25.8 \text{ kN/m}}$$

Centroid of load from outside face of raft

$$y_{corner1} = \mathbf{270 \text{ mm}}$$

Corner load number 2

Load type

Line load in x direction

Dead load

$$w_{Dcorner2} = \mathbf{9.6 \text{ kN/m}}$$

Live load

$$w_{Lcorner2} = \mathbf{0.0 \text{ kN/m}}$$

Ultimate load

$$w_{ultcorner2} = 1.4 \times w_{Dcorner2} + 1.6 \times w_{Lcorner2} = \mathbf{13.4 \text{ kN/m}}$$

Centroid of load from outside face of raft

$$y_{corner2} = \mathbf{75 \text{ mm}}$$

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Corner load number 3

Load type

Dead load

Live load

Ultimate load

Centroid of load from outside face of raft

Line load in y direction

$W_{Dcorner3} = 9.6 \text{ kN/m}$

$W_{Lcorner3} = 0.0 \text{ kN/m}$

$W_{ultcorner3} = 1.4 \times W_{Dcorner3} + 1.6 \times W_{Lcorner3} = 13.4 \text{ kN/m}$

$X_{corner3} = 75 \text{ mm}$

Corner load number 4

Load type

Dead load

Live load

Ultimate load

Centroid of load from outside face of raft

Line load in y direction

$W_{Dcorner4} = 11.5 \text{ kN/m}$

$W_{Lcorner4} = 3.0 \text{ kN/m}$

$W_{ultcorner4} = 1.4 \times W_{Dcorner4} + 1.6 \times W_{Lcorner4} = 20.9 \text{ kN/m}$

$X_{corner4} = 270 \text{ mm}$

Corner bearing pressure check

Total uniform load at formation level

$W_{udlcorner} = W_{Dudl} + W_{Ludl} + W_{edge}/b_{bearing} + W_{hcorethick} = 21.3 \text{ kN/m}^2$

Net bearing press avail to resist line/point loads

$Q_{netcorner} = Q_{allow} - W_{udlcorner} = 28.7 \text{ kN/m}^2$

Total line/point loads

Total unfactored line load in x direction

$W_{\Sigma line x} = 27.6 \text{ kN/m}$

Total ultimate line load in x direction

$W_{\Sigma ultline x} = 39.2 \text{ kN/m}$

Total unfactored line load in y direction

$W_{\Sigma line y} = 24.1 \text{ kN/m}$

Total ultimate line load in y direction

$W_{\Sigma ultline y} = 34.3 \text{ kN/m}$

Total unfactored point load

$W_{\Sigma point} = 0.0 \text{ kN}$

Total ultimate point load

$W_{\Sigma ultpoint} = 0.0 \text{ kN}$

Length of side of sq reqd to resist line/point loads

$p_{corner} = [W_{\Sigma line x} + W_{\Sigma line y} + \sqrt{((W_{\Sigma line x} + W_{\Sigma line y})^2 + 4 \times Q_{netcorner} \times W_{\Sigma point})}] / (2 \times Q_{netcorner})$
 $p_{corner} = 1802 \text{ mm}$

Bending moment about x-axis due to load/reaction eccentricity

Moment due to load 1 (x line)

$M_{x1} = \max(0 \text{ kNm}, W_{ultcorner1} \times p_{corner} \times (p_{corner}/2 - y_{corner1})) = 29.3 \text{ kNm}$

Moment due to load 2 (x line)

$M_{x2} = \max(0 \text{ kNm}, W_{ultcorner2} \times p_{corner} \times (p_{corner}/2 - y_{corner2})) = 20.0 \text{ kNm}$

Total moment about x axis

$M_{\Sigma x} = 49.3 \text{ kNm}$

Bending moment about y-axis due to load/reaction eccentricity

Moment due to load 3 (y line)

$M_{y3} = \max(0 \text{ kNm}, W_{ultcorner3} \times p_{corner} \times (p_{corner}/2 - X_{corner3})) = 20.0 \text{ kNm}$

Moment due to load 4 (y line)

$M_{y4} = \max(0 \text{ kNm}, W_{ultcorner4} \times p_{corner} \times (p_{corner}/2 - X_{corner4})) = 23.8 \text{ kNm}$

Total moment about y axis

$M_{\Sigma y} = 43.8 \text{ kNm}$

Check top reinforcement in edge beams for load/reaction eccentric moment

Max moment due to load/reaction eccentricity

$M_{\Sigma} = \max(M_{\Sigma x}, M_{\Sigma y}) = 49.3 \text{ kNm}$

Assume all of this moment is resisted by edge beam

From edge beam design checks away from corners

Moment due to edge beam spanning depression

$M_{\Sigma edge} = 37.3 \text{ kNm}$

Total moment to be resisted

$M_{\Sigma cornerbp} = M_{\Sigma} + M_{\Sigma edge} = 86.6 \text{ kNm}$

Width of section in compression zone

$b_{edgetop} = b_{edge} + b_{boot} = 625 \text{ mm}$

K factor

$K_{cornerbp} = M_{\Sigma cornerbp} / (f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = 0.011$

Lever arm

$Z_{cornerbp} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{cornerbp}/0.9)}) = 581 \text{ mm}$

Total area of top steel required

$A_{scornerbp} = M_{\Sigma cornerbp} / ((1.0/\gamma_s) \times f_y \times Z_{cornerbp}) = 343 \text{ mm}^2$

PASS - $A_{scornerbp} \leq A_{sedgetop}$ - Area of reinforcement provided to resist eccentric moment is adequate

The allowable bearing pressure at the corner will not be exceeded

Corner beam bending check

Cantilever span of edge beam

$l_{corner} = \phi_{depth} / \sqrt{(2)} + d_{edgetop}/2 = 1614 \text{ mm}$

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Moment and shear due to self weight

Ultimate self weight udl

$$W_{edgeult} = 1.4 \times W_{edge} = 15.7 \text{ kN/m}$$

Average ultimate slab udl (approx)

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

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

Self weight and slab bending moment

$$M_{cornersw} = (W_{edgeult} + W_{cornerslab}) \times l_{corner}^2 / 2 = 20.5 \text{ kNm}$$

Self weight and slab shear force

$$V_{cornersw} = (W_{edgeult} + W_{cornerslab}) \times l_{corner} = 25.4 \text{ kN}$$

Moment and shear due to udl

Maximum ultimate udl

$$W_{cornerudl} = ((1.4 \times W_{Dudl}) + (1.6 \times W_{Ludl})) \times \phi_{depthick} / \sqrt{(2)} = 12.3 \text{ kN/m}$$

Bending moment

$$M_{cornerudl} = W_{cornerudl} \times l_{corner}^2 / 6 = 5.3 \text{ kNm}$$

Shear force

$$V_{cornerudl} = W_{cornerudl} \times l_{corner} / 2 = 9.9 \text{ kN}$$

Moment and shear due to line loads in x direction

Bending moment

$$M_{cornerlinex} = W_{ultlinex} \times l_{corner}^2 / 2 = 51.1 \text{ kNm}$$

Shear force

$$V_{cornerlinex} = W_{ultlinex} \times l_{corner} = 63.3 \text{ kN}$$

Moment and shear due to line loads in y direction

Bending moment

$$M_{cornerliney} = W_{ultliney} \times l_{corner}^2 / 2 = 44.7 \text{ kNm}$$

Shear force

$$V_{cornerliney} = W_{ultliney} \times l_{corner} = 55.4 \text{ kN}$$

Total moments and shears due to point loads

Bending moment about x axis

$$M_{cornerpointx} = 0.0 \text{ kNm}$$

Bending moment about y axis

$$M_{cornerpointy} = 0.0 \text{ kNm}$$

Shear force

$$V_{cornerpoint} = 0.0 \text{ kN}$$

Resultant moments and shears

Total moment about x axis

$$M_{\Sigma cornerx} = M_{cornersw} + M_{cornerudl} + M_{cornerliney} + M_{cornerpointx} = 70.6 \text{ kNm}$$

Total shear force about x axis

$$V_{\Sigma cornerx} = V_{cornersw} + V_{cornerudl} + V_{cornerliney} + V_{cornerpoint} = 90.7 \text{ kN}$$

Total moment about y axis

$$M_{\Sigma cornery} = M_{cornersw} + M_{cornerudl} + M_{cornerlinex} + M_{cornerpointy} = 77.0 \text{ kNm}$$

Total shear force about y axis

$$V_{\Sigma cornery} = V_{cornersw} + V_{cornerudl} + V_{cornerlinex} + V_{cornerpoint} = 98.7 \text{ kN}$$

Deflection of both edge beams at corner will be the same therefore design for average of these moments and shears

Design bending moment

$$M_{\Sigma corner} = (M_{\Sigma cornerx} + M_{\Sigma cornery}) / 2 = 73.8 \text{ kNm}$$

Design shear force

$$V_{\Sigma corner} = (V_{\Sigma cornerx} + V_{\Sigma cornery}) / 2 = 94.7 \text{ kN}$$

Reinforcement required in top of edge beam

K factor

$$K_{corner} = M_{\Sigma corner} / (f_{cu} \times b_{edgetop} \times d_{edgetop}^2) = 0.009$$

Lever arm

$$Z_{corner} = d_{edgetop} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{corner} / 0.9)}) = 581 \text{ mm}$$

Area of steel required for bending

$$A_{scornerbend} = M_{\Sigma corner} / ((1.0 / \gamma_s) \times f_y \times Z_{corner}) = 292 \text{ mm}^2$$

Minimum area of steel required

$$A_{scornermin} = A_{sedgetopmin} = 566 \text{ mm}^2$$

Area of steel required

$$A_{scorner} = \max(A_{scornerbend}, A_{scornermin}) = 566 \text{ mm}^2$$

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

Corner beam shear check

Average web width

$$b_w = b_{edge} + (h_{edge} / \tan(\alpha_{edge})) / 2 = 645 \text{ mm}$$

Applied shear stress

$$v_{corner} = V_{\Sigma corner} / (b_w \times d_{edgetop}) = 0.240 \text{ N/mm}^2$$

Tension steel ratio

$$\rho_{corner} = 100 \times A_{sedgetop} / (b_w \times d_{edgetop}) = 0.153$$

From BS8110-1:1997 - Table 3.8

Design concrete shear strength

$$v_{ccorner} = 0.340 \text{ N/mm}^2$$

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

Link area to spacing ratio required

$$A_{sv_upon_svreqcorner} = 0.4 \text{ N/mm}^2 \times b_w / ((1.0 / \gamma_s) \times f_{ys}) = 0.593 \text{ mm}$$

Link area to spacing ratio provided

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

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PASS - $A_{sv_upon_Svreqcorner} \leq A_{sv_upon_Svprovedge}$ - Shear reinforcement provided in edge beams at corners is adequate

Corner beam deflection check

Basic allowable span to depth ratio	$Ratio_{basiccorner} = 7.0$
Moment factor	$M_{factorcorner} = M_{\Sigma corner} / (b_{edgetop} \times d_{edgetop}^2) = 0.315 \text{ N/mm}^2$
Steel service stress	$f_{scorner} = 2/3 \times f_y \times A_{scorner bend} / A_{sedgetop} = 161.250 \text{ N/mm}^2$
Modification factor	$MF_{corner} = \min(2.0, 0.55 + [(477 \text{ N/mm}^2 - f_{scorner}) / (120 \times (0.9 \text{ N/mm}^2 + M_{factorcorner})]) = 2.000$
Modified allowable span to depth ratio	$Ratio_{allowcorner} = Ratio_{basiccorner} \times MF_{corner} = 14.000$
Actual span to depth ratio	$Ratio_{actualcorner} = l_{corner} / d_{edgetop} = 2.637$

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

Internal beam design checks

Basic loading

Hardcore	$W_{hcorethick} = \gamma_{hcore} \times h_{hcorethick} = 2.7 \text{ kN/m}^2$
Internal beam self weight	$w_{int} = 24 \text{ kN/m}^3 \times h_{int} \times b_{int} = 4.9 \text{ kN/m}$

Internal beam load number 1

Load type	Longitudinal line load
Dead load	$w_{Dint1} = 5.0 \text{ kN/m}$
Live load	$w_{Lint1} = 6.0 \text{ kN/m}$
Ultimate load	$w_{ultint1} = 1.4 \times w_{Dint1} + 1.6 \times w_{Lint1} = 16.6 \text{ kN/m}$
Longitudinal line load width	$b_{int1} = 100 \text{ mm}$
Centroid of load from centreline of beam	$x_{int1} = 0 \text{ mm}$

Internal beam bearing pressure check

Total uniform load at formation level	$w_{udint} = w_{Dudl} + w_{Ludl} + w_{hcorethick} + 24 \text{ kN/m}^3 \times h_{int} = 20.0 \text{ kN/m}^2$
Sum of factored longitud'l and eff'tive line loads	$\Sigma UDL_{int} = 16.6 \text{ kN/m}$
Sum of unfactored longitud'l and eff'tive line loads	$\Sigma UDL_{sint} = 11.0 \text{ kN/m}$

Centroid of loads from centreline of internal beam

Load x distance for internal load 1	$Moment_{int1} = w_{ultint1} \times x_{int1} = 0.0 \text{ kN}$
Sum of load x distances	$\Sigma Moment_{int} = 0.0 \text{ kN}$
Centroid of loads	$x_{barint} = \Sigma Moment_{int} / \Sigma UDL_{int} = 0.0 \text{ mm}$
Moment due to eccentricity to be resisted by slab	$M_{eccint} = \Sigma UDL_{int} \times \text{abs}(x_{barint}) = 0.0 \text{ kNm/m}$
Assume moment due to eccentricity is resisted equally by top steel of slab on one side and bottom steel of slab on other	
From slab bending check	

Moment due to depression under slab (hogging)	$M_{\Sigma e} = 1.7 \text{ kNm/m}$
Total moment to be resisted by slab top steel	$M_{slabtopint} = M_{eccint} / 2 + M_{\Sigma e} = 1.7 \text{ kNm/m}$
K factor	$K_{slabtopint} = M_{slabtopint} / (f_{cu} \times d_{slabmin}^2) = 0.005$
Lever arm	$Z_{slabtopint} = d_{slabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabtopint} / 0.9)}) = 95 \text{ mm}$
Area of steel required	$A_{sslabtopintreq} = M_{slabtopint} / ((1.0 / \gamma_s) \times f_{yslab} \times Z_{slabtopint}) = 42 \text{ mm}^2/\text{m}$

PASS - $A_{sslabtopintreq} \leq A_{sslabtop}$ - Area of reinforcement in top of slab is adequate to transfer moment into slab

Mt to be resisted by slab btm stl due to load ecc'ty	$M_{slabbtmint} = M_{eccint} / 2 = 0.0 \text{ kNm/m}$
K factor	$K_{slabbtmint} = M_{slabbtmint} / (f_{cu} \times d_{slabmin}^2) = 0.000$
Lever arm	$Z_{slabbtmint} = d_{slabmin} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{slabbtmint} / 0.9)}) = 95 \text{ mm}$
Area of steel required in bottom	$A_{sslabbtmintreq} = M_{slabbtmint} / ((1.0 / \gamma_s) \times f_{yslab} \times Z_{slabbtmint}) = 0 \text{ mm}^2/\text{m}$

PASS - $A_{sslabbtmintreq} \leq A_{sslabbtm}$ - Area of reinforcement in bottom of slab is adequate to transfer moment into slab

Bearing pressure

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Initially check bearing pressure based on beam soffit/chamfer width only

Allowable bearing width	$b_{\text{bearint}} = b_{\text{int}} = 450 \text{ mm}$
Bearing pressure due to line/point loads	$q_{\text{linepointint}} = \Sigma UDLs_{\text{int}}/b_{\text{bearint}} = 24.4 \text{ kN/m}^2$
Total applied bearing pressure	$q_{\text{int}} = q_{\text{linepointint}} + w_{\text{udlint}} = 44.4 \text{ kN/m}^2$
PASS - $q_{\text{int}} \leq q_{\text{allow}}$ - Allowable bearing pressure is not exceeded	

Internal beam bending check

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

Span of internal beam	$l_{\text{int}} = \phi_{\text{depththick}} + d_{\text{inttop}} = 2237 \text{ mm}$
Ultimate self weight udl	$w_{\text{intult}} = 1.4 \times w_{\text{int}} = 6.8 \text{ kN/m}$
Ultimate slab udl (approx)	$w_{\text{intslab}} = \max(0 \text{ kN/m}, 1.4 \times w_{\text{slab}} \times ((\phi_{\text{depththick}} \times 3/4) - b_{\text{int}})) = 4.7 \text{ kN/m}$
Self weight and slab bending moment	$M_{\text{intsw}} = (w_{\text{intult}} + w_{\text{intslab}}) \times l_{\text{int}}^2/\beta_{\text{udl}} = 5.8 \text{ kNm}$
Self weight shear force	$V_{\text{intsw}} = (w_{\text{intult}} + w_{\text{intslab}}) \times l_{\text{int}}/2 = 12.9 \text{ kN}$

Moments due to applied uniformly distributed loads

Ultimate udl (approx)	$w_{\text{intudl}} = w_{\text{udlult}} \times \phi_{\text{depththick}} \times 3/4 = 13.0 \text{ kN/m}$
Bending moment	$M_{\text{intudl}} = w_{\text{intudl}} \times l_{\text{int}}^2/\beta_{\text{udl}} = 6.5 \text{ kNm}$
Shear force	$V_{\text{intudl}} = w_{\text{intudl}} \times l_{\text{int}}/2 = 14.6 \text{ kN}$

Moment and shear due to load number 1

Bending moment	$M_{\text{int1}} = w_{\text{ultint1}} \times l_{\text{int}}^2/\beta_{\text{udl}} = 8.3 \text{ kNm}$
Shear force	$V_{\text{int1}} = w_{\text{ultint1}} \times l_{\text{int}}/2 = 18.6 \text{ kN}$

Resultant moments and shears

Total moment (hogging and sagging)	$M_{\Sigma \text{int}} = 20.6 \text{ kNm}$
Maximum shear force	$V_{\Sigma \text{int}} = 46.1 \text{ kN}$

Reinforcement required in top

Width of section in compression zone	$b_{\text{inttop}} = b_{\text{int}} = 450 \text{ mm}$
Average web width	$b_{\text{wint}} = b_{\text{int}} = 450 \text{ mm}$
K factor	$K_{\text{inttop}} = M_{\Sigma \text{int}}/(f_{\text{cu}} \times b_{\text{inttop}} \times d_{\text{inttop}}^2) = 0.009$
Lever arm	$z_{\text{inttop}} = d_{\text{inttop}} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{\text{inttop}}/0.9)}) = 368 \text{ mm}$
Area of steel required for bending	$A_{\text{asinttopbend}} = M_{\Sigma \text{int}}/((1.0/\gamma_s) \times f_y \times z_{\text{inttop}}) = 129 \text{ mm}^2$
Minimum area of steel	$A_{\text{asinttopmin}} = 0.0013 \times b_{\text{wint}} \times h_{\text{int}} = 263 \text{ mm}^2$
Area of steel required	$A_{\text{asinttopreq}} = \max(A_{\text{asinttopbend}}, A_{\text{asinttopmin}}) = 263 \text{ mm}^2$
PASS - $A_{\text{asinttopreq}} \leq A_{\text{asinttop}}$ - Area of reinforcement provided in top of internal beams is adequate	

Reinforcement required in bottom

Width of section in compression zone	$b_{\text{intbtm}} = b_{\text{int}} + 0.2 \times l_{\text{int}} = 897 \text{ mm}$
K factor	$K_{\text{intbtm}} = M_{\Sigma \text{int}}/(f_{\text{cu}} \times b_{\text{intbtm}} \times d_{\text{intbtm}}^2) = 0.004$
Lever arm	$z_{\text{intbtm}} = d_{\text{intbtm}} \times \min(0.95, 0.5 + \sqrt{(0.25 - K_{\text{intbtm}}/0.9)}) = 377 \text{ mm}$
Area of steel required for bending	$A_{\text{asintbtmbend}} = M_{\Sigma \text{int}}/((1.0/\gamma_s) \times f_y \times z_{\text{intbtm}}) = 126 \text{ mm}^2$
Minimum area of steel required	$A_{\text{asintbtmmin}} = 0.0013 \times 1.0 \times b_{\text{wint}} \times h_{\text{int}} = 263 \text{ mm}^2$
Area of steel required	$A_{\text{asintbtmreq}} = \max(A_{\text{asintbtmbend}}, A_{\text{asintbtmmin}}) = 263 \text{ mm}^2$
PASS - $A_{\text{asintbtmreq}} \leq A_{\text{asintbtm}}$ - Area of reinforcement provided in bottom of internal beams is adequate	

Internal beam shear check

Applied shear stress	$v_{\text{int}} = V_{\Sigma \text{int}}/(b_{\text{wint}} \times d_{\text{inttop}}) = 0.264 \text{ N/mm}^2$
Tension steel ratio	$\rho_{\text{int}} = 100 \times A_{\text{asinttop}}/(b_{\text{wint}} \times d_{\text{inttop}}) = 0.346$
From BS8110-1:1997 - Table 3.8	

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Design concrete shear strength

$$v_{cint} = \mathbf{0.497 \text{ N/mm}^2}$$

$$v_{int} \leq v_{cint} + \mathbf{0.4 \text{ N/mm}^2} - \text{Therefore minimum links required}$$

Link area to spacing ratio required

$$A_{sv_upon_svreqint} = 0.4 \text{ N/mm}^2 \times b_{wint} / ((1.0/\gamma_s) \times f_{ys}) = \mathbf{0.414 \text{ mm}}$$

Link area to spacing ratio provided

$$A_{sv_upon_svprovint} = N_{intlink} \times \pi \times \phi_{intlink}^2 / (4 \times s_{vint}) = \mathbf{0.628 \text{ mm}}$$

PASS - $A_{sv_upon_svreqint} \leq A_{sv_upon_svprovint}$ - Shear reinforcement provided in internal beams is adequate

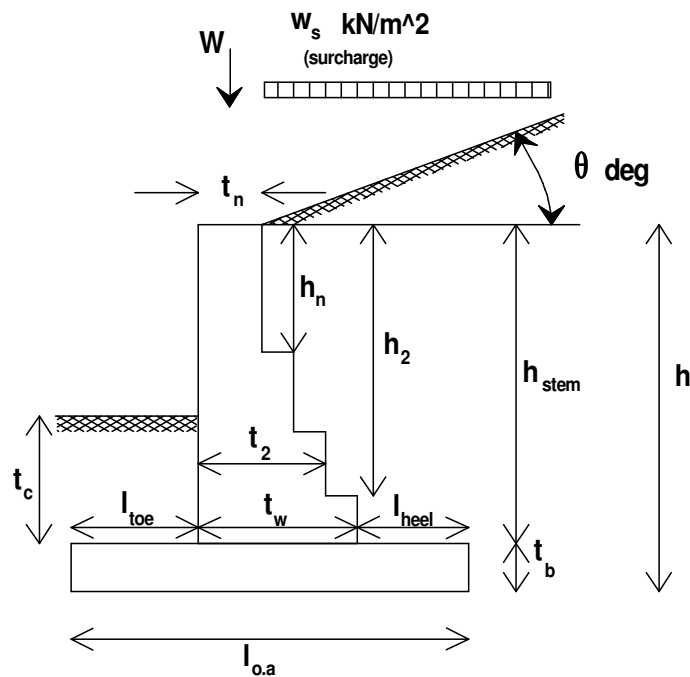
RETAINING WALL CALCULATIONS

TO BE READ IN CONJUNCTION WITH STRUCTURAL REFERENCES AND ARCHITECTS DRAWINGS

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LB STRUCTURAL DESIGN
21 HEOL STRADLING
COITY, BRIDGEND
CF35 6AN

Project Gethin House				Job no. LS378	
Calcs for Garden Retaining Wall				Start page no./Revision 63	
Calcs by LB	Calcs date 19/12/2019	Checked by LB	Checked date 28/01/2020	Approved by	Approved date



CANTILEVER MASONRY RETAINING WALL DESIGN (BS8110&BS5628) MASONRY CANTILEVER RETAINING WALL DESIGN TO CP2 & BS5628 (PART 1)

RETAINING WALL PRESSURES - GRANULAR FILL - NO WALL FRICTION – ASSUMED FULLY DRAINED

General Soil Data

Density of retained soil	$\gamma_s = 18.0 \text{ kN/m}^3$
Angle of internal friction	$\phi = 28.0^\circ$
Angle of soil surface	$\theta = 0.0^\circ$
Ground cover over base at front of wall – if > 0mm, assumed permanent	$t_c = 200 \text{ mm}$

Loads on wall and ground

Surcharge load (horizontal plan area)	$w_s = 1.5 \text{ kN/m}^2$
Design vertical dead load on wall (unfactored)	$W_v = 0.00 \text{ kN/m}$
Design vertical live load on wall (unfactored)	$W_l = 0.00 \text{ kN/m}$

Stem details

Density of wall stem brickwork	$\gamma_w = 19.0 \text{ kN/m}^3$
Wall base section thickness	$t_w = 1115.0 \text{ mm}$
Height retained	$h = 1725 \text{ mm}$

Base details

Density of wall base	$\gamma_b = 23.6 \text{ kN/m}^3$
Length of heel	$l_{heel} = 200 \text{ mm}$

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Length of toe $l_{toe} = 200 \text{ mm}$

Base thickness $t_b = 225 \text{ mm}$

Length of base $l_{o_a} = l_{toe} + t_w + l_{heel} = 1515 \text{ mm}$

Height of active soil (at back of stem and heel)

Height of wall stem $h_{stem} = h - t_b = 1500 \text{ mm}$

$h_{soil} = h + l_{heel} \times \tan(\theta) = 1725 \text{ mm}$

Load factors

Dead Load Factor $\gamma_{f_d} = 1.40$

Imposed Load Factor $\gamma_{f_i} = 1.60$

Coefficients using Rankine

Active coefficient (Sloping Surface) - no friction

$$k_{a_s} = (\cos(\theta) - \sqrt{((\cos(\theta))^2 - (\cos(\phi))^2)}) / (\cos(\theta) + \sqrt{((\cos(\theta))^2 - (\cos(\phi))^2)}) = 0.361$$

Active coefficient (level surface)

$$k_a = (1 - \sin(\phi)) / (1 + \sin(\phi)) = 0.361$$

Passive coefficient (level surface)

$$k_p = (1 + \sin(\phi)) / (1 - \sin(\phi)) = 2.770$$

At - rest coefficient

$$k_o = (1 - \sin(\phi)) = 0.531$$

For a one metre width - (Horizontal Components - Active Earth Pressures)

At the virtual back of the wall (back of heel)

Surcharge

pressure $p_s = k_{a_s} \times w_s = 0.5 \text{ kN/m}^2$

force – horizontal $F_s = (p_s \times h_{soil}) \times (\cos(\theta))^2 = 0.9 \text{ kN/m}$

overturning moment about toe $M_s = F_s \times h_{soil} / 2 = 0.8 \text{ kNm/m}$

Backfill

pressure $p_a = k_{a_s} \times \gamma_s \times h_{soil} = 11.2 \text{ kN/m}^2$

force – horizontal $F_a = (p_a \times h_{soil} / 2) \times (\cos(\theta))^2 = 9.7 \text{ kN/m}$

overturning moment about the toe $M_a = F_a \times h_{soil} / 3 = 5.6 \text{ kNm/m}$

Total horizontal force (unfactored)

$$F_t = F_s + F_a = 10.6 \text{ kN/m}$$

Total horizontal force (factored)

$$F_{t_ult} = \gamma_{f_i} \times F_s + \gamma_{f_d} \times F_a = 15.0 \text{ kN/m}$$

Total overturning moment (unfactored)

$$M_t = M_s + M_a = 6.4 \text{ kNm/m}$$

Total overturning moment (factored)

$$M_{t_ult} = \gamma_{f_i} \times M_s + \gamma_{f_d} \times M_a = 9.1 \text{ kNm/m}$$

Average load factor

$$\gamma_{f_av} = M_{t_ult} / M_t = 1.43$$

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STABILITY CHECKS

(Unfactored Loads - Active Soil Pressures)

1) SLIDING

Coefficient of friction $\mu = \tan(\phi) \times \text{factor} = \mathbf{0.399}$ factor = **0.75**

Passive pressures in front of wall

Top of base $pp_1 = k_p \times t_c \times \gamma_s = \mathbf{10.0}$ kN/m²

Underside of base $pp_2 = k_p \times (t_c + t_b) \times \gamma_s = \mathbf{21.2}$ kN/m²

Total resistance

Base $PP_{\text{base}} = (pp_1 + pp_2) \times t_b / 2 = \mathbf{3.5}$ kN/m

Passive resistance **Soil in front of wall assumed to resist sliding**

$P = PP_{\text{base}} \times \text{soil_resists} = \mathbf{3.5}$ kN/m

Factor of safety against sliding

Factor of safety required $FOS_{\text{reqd sliding}} = \mathbf{2.00}$

Length of heel provided $l_{\text{heel}} = \mathbf{200}$ mm

Conservatively, it is assumed that the surcharge load does not provide downward pressure on the heel in the sliding calculation.

$FOS_{\text{sliding}} = (((l_{\text{heel}} \times (\gamma_s \times (h_{\text{stem}} + l_{\text{heel}} \times \tan(\theta)/2) + \gamma_b \times t_b)) + l_{\text{toe}} \times \gamma_b \times t_b + W_v + (t_w \times (\gamma_w \times h_{\text{stem}} + \gamma_b \times t_b))) \times \mu) + P) / F_t$

$FOS_{\text{sliding}} = \mathbf{2.031}$

Factor of safety against sliding = 2.0 i.e. > 2.0 - OK

2) OVERTURNING

Factor of safety against overturning

Factor of safety required $FOS_{\text{reqd o_turn}} = \mathbf{2.00}$

Length of toe provided $l_{\text{toe}} = \mathbf{200}$ mm

Conservatively, it is assumed that the surcharge load does not provide downward pressure on the heel in the overturning calculation.

Taking Moments about front of the toe

$M_{r1} = (W_v + h_{\text{stem}} \times t_w \times \gamma_w) \times (l_{\text{toe}} + t_w / 2) = \mathbf{24.1}$ kNm/m

$M_{r2} = l_{\text{heel}} \times h_{\text{stem}} \times \gamma_s \times (l_{\text{toe}} + t_w + l_{\text{heel}} / 2) = \mathbf{7.6}$ kNm/m

$M_{r3} = (l_{\text{toe}} + t_w + l_{\text{heel}})^2 \times t_b \times \gamma_b / 2 = \mathbf{6.1}$ kNm/m

$M_{r4} = l_{\text{heel}} \times l_{\text{heel}} \times \tan(\theta) / 2 \times \gamma_s \times (l_{\text{toe}} + t_w + l_{\text{heel}} \times 2/3) = \mathbf{0.0}$ kNm/m

Resisting moment $M_r = M_{r1} + M_{r2} + M_{r3} + M_{r4} = \mathbf{37.8}$ kNm/m

Factor of safety against overturning $FOS_{\text{o_turn}} = M_r / \text{abs}(M_t) = \mathbf{5.94}$

Factor of safety against overturning = 5.9 i.e. > 2.0 - OK

Overall length of base $l_{\text{o_a}} = l_{\text{toe}} + t_w + l_{\text{heel}} = \mathbf{1.515}$ m

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3) BEARING PRESSURE (INCLUDING LIVE LOADS AND TOE COVER)

Total overturning moment for bearing calculation $M_t = 6.4 \text{ kNm/m}$

Total resisting moment for bearing calculation $M_r = 37.8 \text{ kNm/m}$

Total vertical load on base

$$N = t_w \times h_{stem} \times \gamma_w + (h_{stem} + l_{heel} \times \tan(\theta)/2) \times l_{heel} \times \gamma_s + t_c \times l_{toe} \times \gamma_s + l_{o_a} \times t_b \times \gamma_b + W_v + w_s \times l_{heel} + W_l$$

$$N = 46.2 \text{ kN/m}$$

Eccentricity of vertical load

$$e_1 = (M_r - \text{abs}(M_t) + t_c \times l_{toe}^2 \times \gamma_s/2 + w_s \times l_{heel} \times (l_{toe} + t_w + l_{heel}/2) + W_l \times (l_{toe} + t_w/2)) / N = 0.691 \text{ m}$$

$$e = l_{o_a} / 2 - e_1 = 0.067 \text{ m}$$

Resultant lies inside middle third

Maximum bearing pressure $p_{max} = \text{if}(\text{abs}(e) < (l_{o_a}/6), \sigma_{max2}, \sigma_{max1}) = 38.6 \text{ kN/m}^2$

Permissible bearing pressure $p_{bearing} = 100 \text{ kN/m}^2$

Maximum bearing pressure is 39 kN/m² i.e. < Allowable - OK

DESIGN BASE PRESSURES BASED UPON AT REST EARTH PRESSURES

For a one metre width - (Horizontal Components - At Rest Earth Pressures)

Surcharge

pressure $p_{so} = k_o \times w_s = 0.8 \text{ kN/m}^2$

force $F_{so} = p_{so} \times h = 1.4 \text{ kN/m}$

moment $M_{so} = F_{so} \times h / 2 = 1.2 \text{ kNm/m}$

Backfill

pressure $p_o = k_o \times \gamma_s \times h = 16.5 \text{ kN/m}^2$

force $F_o = p_o \times h / 2 = 14.2 \text{ kN/m}$

moment $M_o = F_o \times h / 3 = 8.2 \text{ kNm/m}$

Total horizontal force (unfactored) $F_{t_o} = F_{so} + F_o = 15.6 \text{ kN/m}$

Total horizontal force (factored) $F_{t_ult_o} = \gamma_{f_i} \times F_{so} + \gamma_{f_d} \times F_o = 22.1 \text{ kN/m}$

Total moment (unfactored) $M_{t_o} = M_{so} + M_o = 9.4 \text{ kNm/m}$

Total horizontal force (factored) $M_{t_ult_o} = \gamma_{f_i} \times M_{so} + \gamma_{f_d} \times M_o = 13.3 \text{ kNm/m}$

Average load factor $\gamma_{f_av} = M_{t_ult_o} / M_{t_o} = 1.43$

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Total vertical load on base

$$N = t_w \times h_{\text{stem}} \times \gamma_w + (h_{\text{stem}} + l_{\text{heel}} \times \tan(\theta) / 2) \times l_{\text{heel}} \times \gamma_s + t_c \times l_{\text{toe}} \times \gamma_s + l_{o_a} \times t_b \times \gamma_b + W_v + W_s \times l_{\text{heel}} + W_l$$

$$N = 46.2 \text{ kN/m}$$

Total resistance moment (no live loads included or toe cover)

$$M_r = 37.806 \text{ kNm/m}$$

Eccentricity of vertical load

$$e_1 = (M_r - M_{t_o} + t_c \times l_{\text{toe}}^2 \times \gamma_s / 2 + W_s \times l_{\text{heel}} \times (l_{\text{toe}} + t_w + l_{\text{heel}} / 2) + W_l \times (l_{\text{toe}} + t_w / 2)) / N = 0.626 \text{ m}$$

$$e = l_{o_a} / 2 - e_1 = 0.131 \text{ m}$$

Resultant lies inside middle third

1) Resultant outside middle third **This Section is Invalid as Resultant is within Middle Third**

$$\sigma_{\text{max}1} = (2 \times N) / (3 \times (l_{o_a} / 2 - \text{abs}(e))) = 49 \text{ kN/m}^2$$

$$l_{b1} = (N \times 2 / \sigma_{\text{max}1}) = 1.878 \text{ m}$$

2) Resultant inside middle third

$$\sigma_{\text{max}2} = N / l_{o_a} + 6 \times N \times \text{abs}(e) / l_{o_a}^2 = 46 \text{ kN/m}^2$$

$$\sigma_{\text{min}2} = N / l_{o_a} - 6 \times N \times \text{abs}(e) / l_{o_a}^2 = 15 \text{ kN/m}^2$$

$$l_{b2} = l_{o_a} = 1.515 \text{ m}$$

Pressures under the base

$$\sigma_{\text{max}} = \text{if}(\text{abs}(e) < l_{o_a} / 6, \sigma_{\text{max}2}, \sigma_{\text{max}1}) = 46.4 \text{ kN/m}^2$$

$$\sigma_{\text{min}} = \text{if}(\text{abs}(e) < l_{o_a} / 6, \sigma_{\text{min}2}, 0 \text{ kN/m}^2) = 14.6 \text{ kN/m}^2$$

Bearing length

$$l_p = \text{if}(\text{abs}(e) < (l_{o_a} / 6), l_{b2}, l_{b1}) = 1.515 \text{ m}$$

Pressures under the heel/stem interface

$$l_{p_heel} = l_p - l_{\text{toe}} - t_w = 0.200 \text{ m}$$

$$\sigma_{\text{heel}} = \max(((\sigma_{\text{max}} - \sigma_{\text{min}}) \times l_{p_heel} / l_p) + \sigma_{\text{min}}, 0 \text{ kN/m}^2) = 18.8 \text{ kN/m}^2$$

Pressures under the toe/stem interface

$$\sigma_{\text{toe}} = ((\sigma_{\text{max}} - \sigma_{\text{min}}) \times (l_{p_heel} + t_w) / l_p) + \sigma_{\text{min}} = 42.2 \text{ kN/m}^2$$

BASE DESIGN - HEEL

Shears and Moments

$$\text{Vertical load downward } W_{\text{ult_heel}} = (h_{\text{stem}} \times \gamma_s + \gamma_b \times t_b) \times \gamma_{f_d} + W_s \times \gamma_{f_i} = 47.6 \text{ kN/m}^2$$

$$W_{\text{ult_heel}1} = (l_{\text{heel}} \times \tan(\theta) / 2 \times \gamma_s) \times \gamma_{f_d} = 0.0 \text{ kN/m}^2$$

Ultimate Shear (at wall face)

$$\text{downward } V_{\text{ult_pos_heel}} = (W_{\text{ult_heel}} + W_{\text{ult_heel}1}) \times l_{\text{heel}} = 9.5 \text{ kN/m}$$

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upward

$$V_{ult_neg_heel} = (\sigma_{min} \times \max(l_{p_heel}, 0 \text{ mm}) + (\sigma_{heel} - \sigma_{min}) \times (\max(l_{p_heel}, 0 \text{ mm}))^2 / (2 \times l_{p_heel})) \times \gamma_{f_av}$$

$$V_{ult_neg_heel} = \mathbf{4.8 \text{ kN/m}}$$

Nett Ultimate Shear

$$V_{ult_heel} = \text{abs}(V_{ult_pos_heel} - V_{ult_neg_heel}) = \mathbf{4.8 \text{ kN/m}}$$

Ultimate Moment (at centre of wall)

$$\text{clockwise } M_{ult_pos_heel} = w_{ult_heel} \times l_{heel} \times (l_{heel} + t_w) / 2 + w_{ult_heel1} \times l_{heel} \times (2 \times l_{heel} / 3 + t_w / 2) = \mathbf{6.3 \text{ kNm/m}}$$

anticlockwise

$$M_{ult_neg_heel} = ((\sigma_{min} \times l_{p_heel} \times (l_{p_heel} + t_w) / 2) + (\sigma_{heel} - \sigma_{min}) / 2 \times l_{p_heel} \times (l_{p_heel} / 3 + t_w / 2)) \times \gamma_{f_av}$$

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

Nett Ultimate Moment

$$M_{ult_heel} = \text{abs}(M_{ult_pos_heel} - M_{ult_neg_heel}) = \mathbf{3.1 \text{ kNm/m}}$$

HEEL REINFORCEMENT - MESH

Tension Reinforcement

Cover to bars in the top of the heel

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

Effective depth to bars

$$d_{heel} = t_b - C_{r_heel} - (6 \text{ mm}) = \mathbf{184 \text{ mm}}$$

Assume the base is a cantilevered slab

Characteristic strength of reinforcement

$$f_y = \mathbf{460 \text{ N/mm}^2}$$

Characteristic strength of concrete

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

Area of reinforcement required

$$K_{heel} = \text{abs}(M_{ult_heel}) / (d_{heel}^2 \times f_{cu}) = \mathbf{0.003} \quad K' = 0.156$$

Compression steel not required

Lever arm

$$z_{heel} = \min((0.95 \times d_{heel}), (d_{heel} \times (0.5 + \sqrt{(0.25 - K_{heel} / 0.9)}))) = \mathbf{175 \text{ mm}}$$

Reinforcement required

$$A_{s_heel} = \text{abs}(M_{ult_heel}) / (1 / \gamma_{ms} \times f_y \times z_{heel}) = \mathbf{41 \text{ mm}^2/\text{m}}$$

Minimum reinforcement $A_{s_min_heel} = \text{if}(f_y == 460 \text{ N/mm}^2, 0.0013 \times t_b, 0.0024 \times t_b) = \mathbf{293 \text{ mm}^2/\text{m}}$

$$A_{s_req_heel} = \max(A_{s_heel}, A_{s_min_heel}) = \mathbf{293 \text{ mm}^2/\text{m}}$$

Tension steel provided

$$\text{Use A393 Mesh } A_{s_prov_heel} = A_{sl_heel} = \mathbf{393 \text{ mm}^2/\text{m}}$$

Area of main tension steel provided in heel sufficient

Heel - Check min and max areas of steel

Total area of concrete

$$A_c = t_b = \mathbf{225000 \text{ mm}^2/\text{m}}$$

Minimum percentage area of reinforcement $k = \mathbf{0.13 \%}$

Minimum area of steel required

$$A_{s_min} = k \times A_c = \mathbf{293 \text{ mm}^2/\text{m}}$$

Maximum area of steel allowed

$$A_{s_max} = 4 \% \times A_c = \mathbf{9000 \text{ mm}^2/\text{m}}$$

Area of main steel provided OK

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Heel Distribution Steel

Minimum percentage of distribution reinforcement $k_1 = 0.13 \%$

Minimum Area Required $A_{s_dist_req} = k_1 \times A_c = 293 \text{ mm}^2/\text{m}$

Area Provided $A_{s_dist_base} = A_{st_heel} = 393 \text{ mm}^2/\text{m}$

Area of distribution steel provided OK

CHECK OF NOMINAL COVER

Cover to outer tension reinforcement

$$C_{heel} = t_b - d_{heel} - d_{sl_heel} / 2 = 36.0 \text{ mm}$$

Permissible minimum nominal cover to all reinforcement (Table 3.4)

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

Cover over outer steel OK (top of heel)

HEEL SHEAR RESISTANCE – AT WALL FACE

Applied shear stress at wall face

$$v_{heel} = V_{ult_heel} / d_{heel} = 0.03 \text{ N/mm}^2$$

Check shear stress to clause 3.4.5.2 (BS8110:Pt 1)

$$V_{allowable} = \min ((0.8 \text{ N}^{1/2}/\text{mm}) \times \sqrt{f_{cu}}, 5 \text{ N/mm}^2) = 4.73 \text{ N/mm}^2$$

Shear stress < maximum - OK

Shear stresses to clause 3.4.5.4 (BS8110:Pt1)

Allowable shear stress

$$v_{c_heel} = 0.51 \text{ N/mm}^2$$

No shear reinforcement required

BASE DESIGN - TOE

Shears and Moments

Conservatively all soil overburden on the toe is ignored

Ultimate Shear (conservatively - at face of wall)

$$V_{ult_toe} = ((\sigma_{toe} \times l_{toe}) + (((\sigma_{max} - \sigma_{toe}) / 2) \times l_{toe})) \times \gamma_{f_av} - l_{toe} \times t_b \times \gamma_b \times \gamma_{f_d} = 11.1 \text{ kN/m}$$

Ultimate Moment (at centre of wall)

$$M_{ult_toe} = ((\sigma_{toe} \times l_{toe} \times (l_{toe} + t_w)/2) + ((\sigma_{max} - \sigma_{toe})/2 \times l_{toe} \times (2 \times l_{toe}/3 + t_w/2))) \times \gamma_{f_av} - l_{toe} \times t_b \times \gamma_b \times \gamma_{f_d} \times (l_{toe}/2 + t_w/2)$$

$$M_{ult_toe} = 7.3 \text{ kNm/m}$$

TOE REINFORCEMENT - MESH

Tension Reinforcement

Cover to bars in the bottom of the toe $C_{r_toe} = 35 \text{ mm}$

Effective depth to bars $d_{toe} = t_b - C_{r_toe} - (6\text{mm}) = 184 \text{ mm}$

Assume the base is a cantilevered slab

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Characteristic strength of reinforcement $f_y = 460 \text{ N/mm}^2$

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

Area of reinforcement required

$$K_{toe} = \text{abs}(M_{ult_toe}) / (d_{toe}^2 \times f_{cu}) = 0.006 \quad K' = 0.156$$

Compression steel not required

Lever arm $z_{toe} = \min((0.95 \times d_{toe}), (d_{toe} \times (0.5 + \sqrt{(0.25 - K_{toe}/0.9)}))) = 175 \text{ mm}$

Reinforcement required $A_{s_toe} = \text{abs}(M_{ult_toe}) / (1/\gamma_{ms} \times f_y \times z_{toe}) = 96 \text{ mm}^2/\text{m}$

Minimum reinforcement $A_{s_min_toe} = \text{if}(f_y == 460 \text{ N/mm}^2, 0.0013 \times t_b, 0.0024 \times t_b) = 293 \text{ mm}^2/\text{m}$

$A_{s_req_toe} = \max(A_{s_toe}, A_{s_min_toe}) = 293 \text{ mm}^2/\text{m}$

Tension steel provided

Use A393 Mesh $A_{s_prov_toe} = A_{s_toe} = 393 \text{ mm}^2/\text{m}$

Area of main tension steel provided in toe sufficient

Toe - Check min and max areas of steel

Total area of concrete $A_c = t_b = 225000 \text{ mm}^2/\text{m}$

Minimum percentage area of reinforcement $k = 0.13 \%$

Minimum area of steel required $A_{s_min} = k \times A_c = 293 \text{ mm}^2/\text{m}$

Maximum area of steel allowed $A_{s_max} = 4 \% \times A_c = 9000 \text{ mm}^2/\text{m}$

Area of main steel provided OK

Toe Distribution Steel

Minimum percentage of distribution reinforcement $k_1 = 0.13 \%$

Minimum Area Required $A_{s_dist_req} = k_1 \times A_c = 293 \text{ mm}^2/\text{m}$

Area Provided $A_{s_dist_base} = A_{s_toe} = 393 \text{ mm}^2/\text{m}$

Area of distribution steel provided OK

CHECK OF NOMINAL COVER

Cover to outer tension reinforcement

$$c_{toe} = t_b - d_{toe} - d_{sl_toe} / 2 = 36.0 \text{ mm}$$

Permissible minimum nominal cover to all reinforcement (Table 3.4)

$$c_{min_toe} = 35 \text{ mm}$$

Cover over outer steel OK (bottom of toe)

TOE SHEAR RESISTANCE

Applied shear stress

$$V_{toe} = V_{ult_toe} / d_{toe} = 0.06 \text{ N/mm}^2$$

Check shear stress to clause 3.4.5.2 (BS8110:Pt 1)

$$V_{allowable} = \min((0.8 \text{ N}^{1/2}/\text{mm}) \times \sqrt{f_{cu}}, 5 \text{ N/mm}^2) = 4.73 \text{ N/mm}^2$$

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Shear stress < maximum - OK

Shear stresses to clause 3.4.5.4 (BS8110:Pt 1)

Allowable shear stress

$$v_{c_toe} = 0.51 \text{ N/mm}^2$$

No shear reinforcement required

STEM DESIGN

Width of base of wall $t_w = 1115.0 \text{ mm}$

Height of stem of wall $h_{stem} = 1500 \text{ mm}$

DESIGN OF WALL SECTION 1 (AT REST PRESSURES)

Wall base section thickness $t_w = 1115.0 \text{ mm}$

Depth from top of wall to base

$$h_{stem} = 1500 \text{ mm}$$

Applied moment at base of section

$$M_{base} = (k_o \times w_s \times h_{stem}^2 / 2) \times \gamma_{f_i} + (k_o \times \gamma_s \times h_{stem}^3 / 6) \times \gamma_{f_d} = 9.0 \text{ kNm/m}$$

Applied shear force at base of section

$$F_{base} = (k_o \times w_s \times h_{stem}) \times \gamma_{f_i} + (k_o \times \gamma_s \times h_{stem}^2 / 2) \times \gamma_{f_d} = 17.0 \text{ kN/m}$$

CHECK RESISTANCE TO MOMENT WITHOUT FLEXURAL STRENGTH (BS5628:PT 1:CL 36.5.3)

Compressive strength (Table 2) $f_k = 7.00 \text{ N/mm}^2$

Design vertical load per unit length of wall $n_{w_base} = \gamma_{fd} \times (W_v + t_w \times h_{stem} \times \gamma_w) = 28.6 \text{ kN/m}$

Moment of resistance $M_{RC_base} = (n_{w_base} / 2) \times (t_w - (n_{w_base} \times \gamma_m / f_k)) = 15.7 \text{ kNm/m}$

Wall bending - OK

STEM SHEAR (BS5628:PT 1:CL 33)

Mortar designation Mortar = "iii"

Design dead load at the base of the wall $V_{base} = W_v + t_w \times h_{stem} \times \gamma_w = 31.78 \text{ kN/m}$

Design vertical dead load per unit area $g_{A_base} = V_{base} \times \gamma_{fd} / t_w = 0.03 \text{ N/mm}^2$

Characteristic shear strength of masonry

$$f_{v_i} = \min(1.75 \text{ N/mm}^2, 0.35 \text{ N/mm}^2 + (0.6 \times g_{A_base})) = 0.37 \text{ N/mm}^2$$

$$f_{v_iii} = \min(1.4 \text{ N/mm}^2, 0.15 \text{ N/mm}^2 + (0.6 \times g_{A_base})) = 0.17 \text{ N/mm}^2$$

$$f_v = \text{if}((\text{Mortar} == "i") || (\text{Mortar} == "ii"), f_{v_i}, f_{v_iii}) = 0.17 \text{ N/mm}^2$$

Ref BS 5628 Pt 1: Clause 25

Shear stress $v_{base} = F_{base} / t_w = 0.02 \text{ N/mm}^2$

Partial safety factor for shear $\gamma_{mv} = 2.5$

Wall base shear OK

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DESIGN OF WALL SECTION 2 (AT REST PRESSURES)

Wall section thickness $t_{wall_2} = 890.0$ mm

Depth from top of wall to base of section

$h_{wall_2} = 1280$ mm **Warning the height of the section is not a complete number of bricks**

Applied moment at base of section

$$M_{wall_2} = (k_o \times w_s \times h_{wall_2}^2 / 2) \times \gamma_{f_i} + (k_o \times \gamma_s \times h_{wall_2}^3 / 6) \times \gamma_{f_d} = 5.7 \text{ kNm/m}$$

Applied shear force at base of section

$$F_{wall_2} = (k_o \times w_s \times h_{wall_2}) \times \gamma_{f_i} + (k_o \times \gamma_s \times h_{wall_2}^2 / 2) \times \gamma_{f_d} = 12.6 \text{ kN/m}$$

CHECK RESISTANCE TO MOMENT WITHOUT FLEXURAL STRENGTH (BS5628:PT 1:CL 36.5.3)

Compressive strength (Table 2) $f_k = 7.00$ N/mm²

Design vertical load per unit length of wall $n_{w_wall_2} = \gamma_{fd} \times (W_v + t_{wall_2} \times h_{wall_2} \times \gamma_w) = 19.5$ kN/m

Moment of resistance $M_{RC_wall_2} = (n_{w_wall_2} / 2) \times (t_{wall_2} - (n_{w_wall_2} \times \gamma_m / f_k)) = 8.6$ kNm/m

Wall bending - OK

STEM SHEAR (BS5628:PT 1:CL 33)

Mortar designation Mortar = "iii"

Design dead load at the base of the wall $V_{wall_2} = W_v + t_{wall_2} \times h_{wall_2} \times \gamma_w = 21.64$ kN/m

Design vertical dead load per unit area $g_{A_wall_2} = V_{wall_2} \times \gamma_{fd} / t_{wall_2} = 0.02$ N/mm²

Characteristic shear strength of masonry

$$f_{v_i} = \min(1.75 \text{ N/mm}^2, 0.35 \text{ N/mm}^2 + (0.6 \times g_{A_wall_2})) = 0.36 \text{ N/mm}^2$$

$$f_{v_iii} = \min(1.4 \text{ N/mm}^2, 0.15 \text{ N/mm}^2 + (0.6 \times g_{A_wall_2})) = 0.16 \text{ N/mm}^2$$

$$f_v = \text{if}((\text{Mortar} == "i") || (\text{Mortar} == "ii"), f_{v_i}, f_{v_iii}) = 0.16 \text{ N/mm}^2$$

Ref BS 5628 Pt 1: Clause 25

Shear stress $v_{wall_2} = F_{wall_2} / t_{wall_2} = 0.01$ N/mm²

Partial safety factor for shear $\gamma_{mv} = 2.5$

Wall section base shear OK

DESIGN OF WALL SECTION 3 (AT REST PRESSURES)

Wall section thickness $t_{wall_3} = 665.0$ mm

Depth from top of wall to base of section

$h_{wall_3} = 1060$ mm **Warning the height of the section is not a complete number of bricks**

Applied moment at base of section

$$M_{wall_3} = (k_o \times w_s \times h_{wall_3}^2 / 2) \times \gamma_{f_i} + (k_o \times \gamma_s \times h_{wall_3}^3 / 6) \times \gamma_{f_d} = 3.4 \text{ kNm/m}$$

Applied shear force at base of section

$$F_{wall_3} = (k_o \times w_s \times h_{wall_3}) \times \gamma_{f_i} + (k_o \times \gamma_s \times h_{wall_3}^2 / 2) \times \gamma_{f_d} = 8.9 \text{ kN/m}$$

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CHECK RESISTANCE TO MOMENT WITHOUT FLEXURAL STRENGTH (BS5628:PT 1:CL 36.5.3)

Compressive strength (Table 2) $f_k = 7.00 \text{ N/mm}^2$

Design vertical load per unit length of wall $n_{w_wall_3} = \gamma_{fd} \times (W_v + t_{wall_3} \times h_{wall_3} \times \gamma_w) = 12.1 \text{ kN/m}$

Moment of resistance $M_{RC_wall_3} = (n_{w_wall_3} / 2) \times (t_{wall_3} - (n_{w_wall_3} \times \gamma_m / f_k)) = 4.0 \text{ kNm/m}$

Wall bending - OK

STEM SHEAR (BS5628:PT 1:CL 33)

Mortar designation Mortar = "iii"

Design dead load at the base of the wall $V_{wall_3} = W_v + t_{wall_3} \times h_{wall_3} \times \gamma_w = 13.39 \text{ kN/m}$

Design vertical dead load per unit area $g_{A_wall_3} = V_{wall_3} \times \gamma_{fd} / t_{wall_3} = 0.02 \text{ N/mm}^2$

Characteristic shear strength of masonry

$f_{v_i} = \min(1.75 \text{ N/mm}^2, 0.35 \text{ N/mm}^2 + (0.6 \times g_{A_wall_3})) = 0.36 \text{ N/mm}^2$

$f_{v_iii} = \min(1.4 \text{ N/mm}^2, 0.15 \text{ N/mm}^2 + (0.6 \times g_{A_wall_3})) = 0.16 \text{ N/mm}^2$

$f_v = \text{if}((\text{Mortar} == "i")) | | (\text{Mortar} == "ii"), f_{v_i}, f_{v_iii}) = 0.16 \text{ N/mm}^2$

Ref BS 5628 Pt 1: Clause 25

Shear stress $v_{wall_3} = F_{wall_3} / t_{wall_3} = 0.01 \text{ N/mm}^2$

Partial safety factor for shear $\gamma_{mv} = 2.5$

Wall section base shear OK

DESIGN OF WALL SECTION 4 (AT REST PRESSURES)

Wall section thickness $t_{wall_4} = 440.0 \text{ mm}$

Depth from top of wall to base of section

$h_{wall_4} = 620 \text{ mm}$ **Warning the height of the section is not a complete number of bricks**

Applied moment at base of section

$M_{wall_4} = (k_o \times w_s \times h_{wall_4}^2 / 2) \times \gamma_{f_i} + (k_o \times \gamma_s \times h_{wall_4}^3 / 6) \times \gamma_{f_d} = 0.8 \text{ kNm/m}$

Applied shear force at base of section

$F_{wall_4} = (k_o \times w_s \times h_{wall_4}) \times \gamma_{f_i} + (k_o \times \gamma_s \times h_{wall_4}^2 / 2) \times \gamma_{f_d} = 3.4 \text{ kN/m}$

CHECK RESISTANCE TO MOMENT WITHOUT FLEXURAL STRENGTH (BS5628:PT 1:CL 36.5.3)

Compressive strength (Table 2) $f_k = 7.00 \text{ N/mm}^2$

Design vertical load per unit length of wall $n_{w_wall_4} = \gamma_{fd} \times (W_v + t_{wall_4} \times h_{wall_4} \times \gamma_w) = 4.7 \text{ kN/m}$

Moment of resistance $M_{RC_wall_4} = (n_{w_wall_4} / 2) \times (t_{wall_4} - (n_{w_wall_4} \times \gamma_m / f_k)) = 1.0 \text{ kNm/m}$

Wall bending - OK

STEM SHEAR (BS5628:PT 1:CL 33)

Mortar designation Mortar = "iii"

Design dead load at the base of the wall $V_{wall_4} = W_v + t_{wall_4} \times h_{wall_4} \times \gamma_w = 5.18 \text{ kN/m}$

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Design vertical dead load per unit area $g_{A_wall_4} = V_{wall_4} \times \gamma_{fd} / t_{wall_4} = \mathbf{0.01 \text{ N/mm}^2}$

Characteristic shear strength of masonry

$$f_{v_i} = \min(1.75 \text{ N/mm}^2, 0.35 \text{ N/mm}^2 + (0.6 \times g_{A_wall_4})) = \mathbf{0.36 \text{ N/mm}^2}$$

$$f_{v_iii} = \min(1.4 \text{ N/mm}^2, 0.15 \text{ N/mm}^2 + (0.6 \times g_{A_wall_4})) = \mathbf{0.16 \text{ N/mm}^2}$$

$$f_v = \text{if}((\text{Mortar} == "i") || (\text{Mortar} == "ii")), f_{v_i}, f_{v_iii} = \mathbf{0.16 \text{ N/mm}^2}$$

Ref BS 5628 Pt 1: Clause 25

Shear stress

$$v_{wall_4} = F_{wall_4} / t_{wall_4} = \mathbf{0.01 \text{ N/mm}^2}$$

Partial safety factor for shear

$$\gamma_{mv} = \mathbf{2.5}$$

Wall section base shear OK

DESIGN OF WALL SECTION 5 (AT REST PRESSURES)

Wall section thickness $t_{wall_5} = \mathbf{215.0 \text{ mm}}$

Depth from top of wall to base of section

$$h_{wall_5} = \mathbf{300 \text{ mm}}$$

Applied moment at base of section

$$M_{wall_5} = (k_o \times w_s \times h_{wall_5}^2 / 2) \times \gamma_{f_i} + (k_o \times \gamma_s \times h_{wall_5}^3 / 6) \times \gamma_{f_d} = \mathbf{0.1 \text{ kNm/m}}$$

Applied shear force at base of section

$$F_{wall_5} = (k_o \times w_s \times h_{wall_5}) \times \gamma_{f_i} + (k_o \times \gamma_s \times h_{wall_5}^2 / 2) \times \gamma_{f_d} = \mathbf{1.0 \text{ kN/m}}$$

CHECK RESISTANCE TO MOMENT WITHOUT FLEXURAL STRENGTH (BS5628:PT 1:CL 36.5.3)

Compressive strength (Table 2) $f_k = \mathbf{7.00 \text{ N/mm}^2}$

Design vertical load per unit length of wall $n_{w_wall_5} = \gamma_{fd} \times (W_v + t_{wall_5} \times h_{wall_5} \times \gamma_w) = \mathbf{1.1 \text{ kN/m}}$

Moment of resistance $M_{RC_wall_5} = (n_{w_wall_5} / 2) \times (t_{wall_5} - (n_{w_wall_5} \times \gamma_m / f_k)) = \mathbf{0.1 \text{ kNm/m}}$

Wall bending - OK

STEM SHEAR (BS5628:PT 1:CL 33)

Mortar designation Mortar = **"iii"**

Design dead load at the base of the wall $V_{wall_5} = W_v + t_{wall_5} \times h_{wall_5} \times \gamma_w = \mathbf{1.23 \text{ kN/m}}$

Design vertical dead load per unit area $g_{A_wall_5} = V_{wall_5} \times \gamma_{fd} / t_{wall_5} = \mathbf{0.01 \text{ N/mm}^2}$

Characteristic shear strength of masonry

$$f_{v_i} = \min(1.75 \text{ N/mm}^2, 0.35 \text{ N/mm}^2 + (0.6 \times g_{A_wall_5})) = \mathbf{0.35 \text{ N/mm}^2}$$

$$f_{v_iii} = \min(1.4 \text{ N/mm}^2, 0.15 \text{ N/mm}^2 + (0.6 \times g_{A_wall_5})) = \mathbf{0.15 \text{ N/mm}^2}$$

$$f_v = \text{if}((\text{Mortar} == "i") || (\text{Mortar} == "ii")), f_{v_i}, f_{v_iii} = \mathbf{0.15 \text{ N/mm}^2}$$

Ref BS 5628 Pt 1: Clause 25

Shear stress

$$v_{wall_5} = F_{wall_5} / t_{wall_5} = \mathbf{0.00 \text{ N/mm}^2}$$

Partial safety factor for shear

$$\gamma_{mv} = \mathbf{2.5}$$

Wall section base shear OK

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SUMMARY OF RESULTS

Wall Dimensions

Masonry Compressive Strength **Masonry Compressive Strength - 7.0 N/mm²**

Stem Base section **1115mm thick and 1500mm from base to the top of the wall**

Section 2 **890mm thick and 1280mm from base of section 2 to the top of the wall**

Section 3 **665mm thick and 1060mm from base of section 3 to the top of the wall**

Section 4 **440mm thick and 620mm from base of section 4 to the top of the wall**

Section 5 **215mm thick and 300mm from base of section 5 to the top of the wall**

Base Top Reinforcement - Mesh

Heel (Top) Reinforcement **Provide A393 Mesh (Top)**

Base Bottom Reinforcement - Mesh

Toe (Bottom) Reinforcement **Provide A393 Mesh (Bottom)**