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5 Willow Court, Bangor-on-Dee

Extension and Alterations

Structural Calculations

April 2022

5 Willow Court, Bangor-on-Dee

Preamble to Calculations

Initials Date		
Prepared by	AH	04/22
Checked by		

Scope of Calculations

These calculations have been prepared at the request of the owner of the above property. The calculations cover the structural design of roof construction over proposed extension and beams and lintels to support roof, floor and wall construction over openings.

No warranty is given by the issue of the calculations to the structural integrity of any other part of the proposed building for which calculations or structural advice has not been specifically requested or given

Building Regulations Approval

The owners of the property are advised that the approval of the Local Planning and/or Building Control should be obtained prior to any ordering of materials or fabrication. No responsibility is accepted for any changes that may be required as a result of work having proceeded before such approvals have been obtained.

Measurements and Dimensions

Dimensions and details for the purpose of the calculations have been taken from AutoCAD Drawings provided by the Architect for the scheme.

The person responsible for carrying out the Works should ensure that all assumptions and dimensions used in the calculations are checked on site before ordering of materials or fabrication. Any discrepancies or dimensions found to be significantly different from that assumed should be brought to the attention of the undersigned, so that such discrepancy can be checked, and if necessary, the calculations amended.

Design References

Structural design has been based on any or all of the following:

- Building Regulations – Approved Document A
- EN1990 – Basis of Structural Design.
- EN1991 – Actions on Structures.
- EN1992 – Design of Concrete Structures.
- EN1993 – Design of Steel Structures.
- EN1994 – Design of Composite Steel and Concrete Structures.
- EN1995 – Design of Timber Structures.
- EN1996 – Design of Masonry Structures.
- EN1997 – Geotechnical Design

Liability

Nothing in the Contract confers nor purports to confer on any third party any benefit nor any right to enforce any terms of this Contract pursuant to the Contracts (Rights of Third Parties) Act 1999.

Copyright

The design report including any calculations, sketches, drawings and schedules are the copyright of Andrew Hannett Limited who will retain ownership until the account has been settled in full. The information contained herein shall not be shared with or issued to third parties for construction, approvals or any other purposes until the account has been settled in full. Andrew Hannett Limited reserve the right to withdraw the design report at any point until the account has been settled in full.

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CONSTRUCTION (DESIGN & MANAGEMENT) REGULATIONS CDM2015

The design reflected in this design report has been developed for Building Regulations and Construction purposes. Our appointment as Principal Designer stops after issue of this design package.

The Domestic Client duties under CDM2015 must be undertaken by the Contractor, or where there is more than one contractor, the Principal Contractor. Contractors in Domestic Projects must be aware of their Client duties under the Construction (Design and Management) Regulations 2015. Should this not be the case, then further guidance must be sought. Additional guidance in relation to the duties of the various parties is available on the HSE website.

In accordance with the Construction (Design & Management) Regulations 2015 it is our policy to 'Design Out' as many identifiable risks as possible for the construction, maintenance and demolition of the project. A Competent Contractor should be aware of 'normal' risks involved with the construction process. Unusual and/or significant residual risks that cannot be eliminated as part of the design process are listed below:

1. None Identified.

Safe methods and systems of work remain the responsibility of the Principal Contractor and must be identified in the Construction Phase Plan. A Construction Phase Plan must be drawn up for every project by the Contractor or Principal Contractor.

If any party using this design package considers or identifies any 'Abnormal' or 'Unusual' risks that have not been identified above they should share this with the project team immediately.

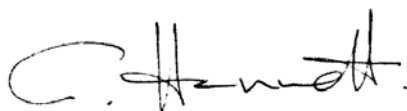
Health and Safety File – when our appointment as Principal Designer ends we will pass the Health and Safety File to the Principal Contractor. The Principal Contractor will be responsible for developing this and handing it to the Client on completion. The CITB has developed an app for iOS and Android platforms to aid in the compilation of Health and Safety Files.

Calculation Pages, Sketches and Drawings contained herein:

Calculation pages 1 to 41 inclusive.

Sketches SK01 – 06 including notes.

Signed:



Date: 29 Apr. 22

Andrew Hannett. B.Eng(Hons), C.Eng, MICE

5 Willow Court, Bangor-on-Dee

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EXTENSION AND ALTERATIONS

Provide calculations in support of structural design of roof construction over proposed extension and beams and lintels to support roof, floor and wall construction over openings.

Loadings

Roof	Slates/Tiles	0.55			
	Battens	0.05	Snow Load = <u>0.75 kN/m²</u>		
	Felt	0.03			
	Trusses	0.15			
		<u>0.78 kN/m²</u>	=	<u>0.78</u>	= <u>0.84 kN/m² (plan)</u>
		(slope)		cos 22	
Roof (warm roof const ⁿ)	Slates/Tiles	0.55			
	Battens	0.05			
	Felt	0.03	Snow Load = <u>0.75 kN/m²</u>		
	Rafters	0.15			
	Insulation	0.05			
	P'board & skim	0.18			
		<u>1.01 kN/m²</u>	=	<u>1.01</u>	= <u>1.05 kN/m² (plan)</u>
		(slope)		cos 15	
Ceiling	Ties	0.10			
	Insulation	0.05	Imposed = <u>0.25 kN/m²</u>		
	P'board & skim	0.18			
		<u>0.33 kN/m²</u>			
Floor	Boards	0.15			
	Joists	0.15	Imposed = <u>1.5 kN/m²</u>		
	Ceiling	0.18			
	Partitions	0.25			
		<u>0.73 kN/m²</u>			
Walls	Block & plaster	<u>1.6 kN/m²</u>			
	Block & 2 plaster	<u>1.8 kN/m²</u>			
	Block & render	<u>1.8 kN/m²</u>			
	Brick	<u>2.1 kN/m²</u>			

Assumptions:

- Roof construction to main part of property comprises trussed rafters spanning between the front and rear elevations.
- First floor construction spans onto rear elevation.

SCHEME

PAGE No.

4

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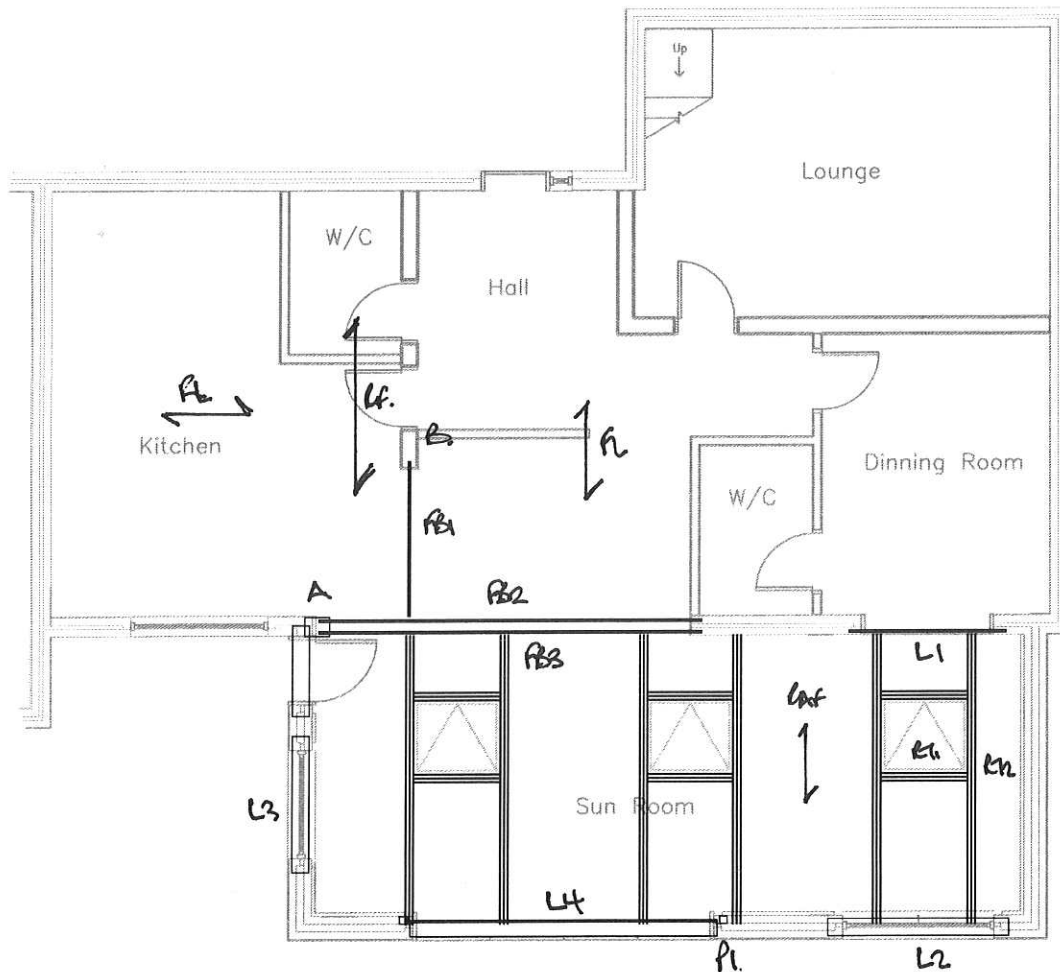
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Roofing R1 Span = 3800mm (plan)

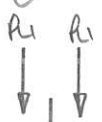
loads to rafters - $g_k = 1.01 - 0.15 = 0.86 \text{ kN/m}^2$

$q_k = 0.75 \text{ kN/m}^2$ $q_k = 0.9 \text{ kN/m}^2$

8/5
-8

Use 50x175 C24 rafters @ 400mm c/c

Roof Truss R1 Span = 1200mm



$R1 - G_k = 1.05 \times 0.4 \times \frac{3}{2} = 0.63 \text{ kN}$
 $q_k = 0.75 \times 0.4 \times \frac{3}{2} = 0.56 \text{ kN}$

$R2 - G_k = 0.9 \text{ kN}$

Use min 50x175 C24 purlin

8/9
-11

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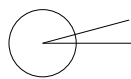
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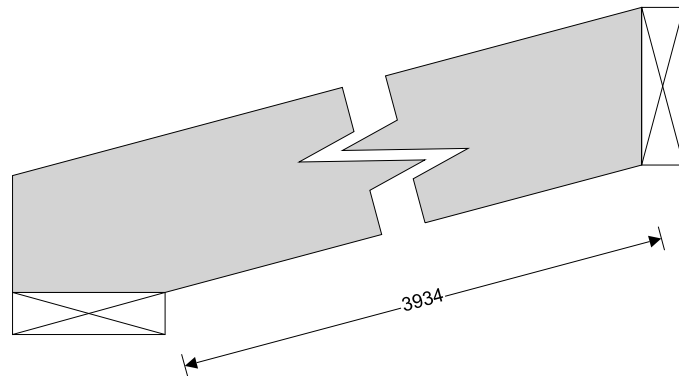
RAFTERS RAF

TIMBER RAFTER DESIGN (BS5268-2:2002)

TEDDS calculation version 1.0.03



15 degrees



Rafter details

Breadth of timber sections

$b = 47 \text{ mm}$

Depth of timber sections

$h = 170 \text{ mm}$

Rafter spacing

$s = 400 \text{ mm}$

Rafter slope

$\alpha = 15.0 \text{ deg}$

Clear span of rafter on horizontal

$L_{clh} = 3800 \text{ mm}$

Clear span of rafter on slope

$L_{cl} = L_{clh} / \cos(\alpha) = 3934 \text{ mm}$

Rafter span

Single span

Timber strength class

C24

Section properties

Cross sectional area of rafter

$A = b \times h = 7990 \text{ mm}^2$

Section modulus

$Z = b \times h^2 / 6 = 226383 \text{ mm}^3$

Second moment of area

$I = b \times h^3 / 12 = 19242583 \text{ mm}^4$

Radius of gyration

$r = \sqrt{I / A} = 49.1 \text{ mm}$

Loading details

Rafter self weight

$F_j = b \times h \times \rho_{char} \times g_{acc} = 0.03 \text{ kN/m}$

Dead load on slope

$F_d = 0.86 \text{ kN/m}^2$

Imposed load on plan

$F_u = 0.75 \text{ kN/m}^2$

Imposed point load

$F_p = 0.90 \text{ kN}$

Modification factors

Section depth factor

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

Load sharing factor

$K_8 = 1.10$

Consider long term load condition

Load duration factor

$K_3 = 1.00$

Total UDL perpendicular to rafter

$F = F_d \times \cos(\alpha) \times s + F_j \times \cos(\alpha) = 0.359 \text{ kN/m}$

Notional bearing length

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

Effective span

$L_{eff} = L_{cl} + L_b = 3940 \text{ mm}$

Check bending stress

Bending stress parallel to grain

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

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Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 8.782 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 3.075 \text{ N/mm}^2$$

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

$$K_{12} = 0.52$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 4.538 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.401 \text{ N/mm}^2$$

PASS - Applied compressive stress within permissible limits**Check combined bending and compressive stress parallel to grain**

Euler stress

$$\sigma_e = \pi^2 \times E_{min} / \lambda^2 = 11.026 \text{ N/mm}^2$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.971$$

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.449 < 1$$

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

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

PASS - Applied shear stress within permissible limits**Check deflection**

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = 11.819 \text{ mm}$$

Bending deflection

$$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = 5.416 \text{ mm}$$

Shear deflection

$$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = 0.155 \text{ mm}$$

Total deflection

$$\delta_{max} = \delta_b + \delta_s = 5.570 \text{ mm}$$

PASS - Total deflection within permissible limits**Consider medium term load condition**

Load duration factor

$$K_3 = 1.25$$

Total UDL perpendicular to rafter

$$F = [F_u \times \cos(\alpha)^2 + F_d \times \cos(\alpha)] \times s + F_j \times \cos(\alpha) = 0.639 \text{ kN/m}$$

Notional bearing length

$$L_b = F \times L_{cl} / [2 \times (b \times \sigma_{cp1} \times K_8 - F)] = 10 \text{ mm}$$

Effective span

$$L_{eff} = L_{cl} + L_b = 3944 \text{ mm}$$

Check bending stress

Bending stress parallel to grain

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

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 10.977 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) = 5.486 \text{ N/mm}^2$$

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

$$K_{12} = 0.47$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 5.147 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) = 0.715 \text{ N/mm}^2$$

PASS - Applied compressive stress within permissible limits

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Check combined bending and compressive stress parallel to grain

Euler stress

$$\sigma_e = \pi^2 \times E_{\min} / \lambda^2 = 11.001 \text{ N/mm}^2$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.954$$

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.663 < 1$$

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

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

Applied shear stress

$$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) = 0.236 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

Check deflection

Permissible deflection

$$\delta_{adm} = 0.003 \times L_{eff} = 11.833 \text{ mm}$$

Bending deflection

$$\delta_b = 5 \times F \times L_{eff}^4 / (384 \times E_{mean} \times I) = 9.684 \text{ mm}$$

Shear deflection

$$\delta_s = 12 \times F \times L_{eff}^2 / (5 \times E_{mean} \times A) = 0.276 \text{ mm}$$

Total deflection

$$\delta_{max} = \delta_b + \delta_s = 9.961 \text{ mm}$$

PASS - Total deflection within permissible limits

Consider short term load condition

Load duration factor

$$K_3 = 1.50$$

Total UDL perpendicular to rafter

$$F = F_d \times \cos(\alpha) \times s + F_l \times \cos(\alpha) = 0.359 \text{ kN/m}$$

Notional bearing length

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

Effective span

$$L_{eff} = L_{cl} + L_b = 3943 \text{ mm}$$

Check bending stress

Bending stress parallel to grain

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

Permissible bending stress

$$\sigma_{m_adm} = \sigma_m \times K_3 \times K_7 \times K_8 = 13.173 \text{ N/mm}^2$$

Applied bending stress

$$\sigma_{m_max} = F \times L_{eff}^2 / (8 \times Z) + F_p \times \cos(\alpha) \times L_{eff} / (4 \times Z) = 6.866 \text{ N/mm}^2$$

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

$$K_{12} = 0.43$$

Permissible compressive stress

$$\sigma_{c_adm} = \sigma_c \times K_3 \times K_8 \times K_{12} = 5.616 \text{ N/mm}^2$$

Applied compressive stress

$$\sigma_{c_max} = F \times L_{eff} \times (\cot(\alpha) + 3 \times \tan(\alpha)) / (2 \times A) + F_p \times \sin(\alpha) / A = 0.431 \text{ N/mm}^2$$

PASS - Applied compressive stress within permissible limits

Check combined bending and compressive stress parallel to grain

Euler stress

$$\sigma_e = \pi^2 \times E_{\min} / \lambda^2 = 11.006 \text{ N/mm}^2$$

Euler coefficient

$$K_{eu} = 1 - (1.5 \times \sigma_{c_max} \times K_{12} / \sigma_e) = 0.975$$

Combined axial compression and bending check

$$\sigma_{m_max} / (\sigma_{m_adm} \times K_{eu}) + \sigma_{c_max} / \sigma_{c_adm} = 0.611 < 1$$

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

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

Applied shear stress

$$\tau_{max} = 3 \times F \times L_{eff} / (4 \times A) + 3 \times F_p \times \cos(\alpha) / (2 \times A) = 0.296 \text{ N/mm}^2$$

PASS - Applied shear stress within permissible limits

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Check deflection

Permissible deflection

$\delta_{adm} = 0.003 \times L_{eff} = 11.830 \text{ mm}$

Bending deflection

$\delta_b = L_{eff}^3 \times (5 \times F \times L_{eff} / 384 + F_p \times \cos(\alpha) / 48) / (E_{mean} \times I) = 10.778 \text{ mm}$

Shear deflection

$\delta_s = 12 \times L_{eff} \times (F \times L_{eff} + 2 \times F_p \times \cos(\alpha)) / (5 \times E_{mean} \times A) = 0.346 \text{ mm}$

Total deflection

$\delta_{max} = \delta_b + \delta_s = 11.124 \text{ mm}$

PASS - Total deflection within permissible limits

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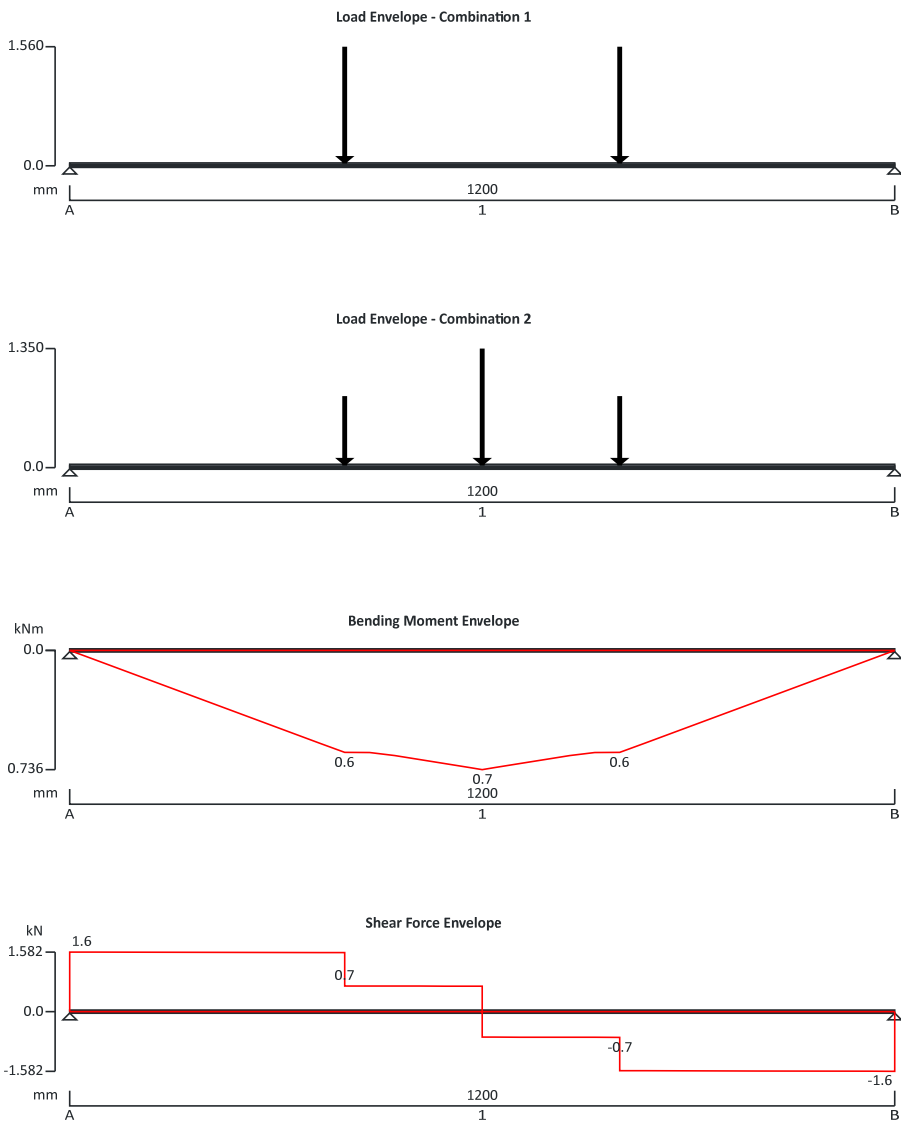
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ROOF TRIMMER RT1

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.04



Applied loading

Beam loads

PL1 Gk	Permanent self weight of beam $\times 1$
PL1 Qk	Permanent point load 0.600 kN at 400 mm
PL1 Gk	Variable UDL point load 0.500 kN at 400 mm
PL1 Qk	Permanent point load 0.600 kN at 800 mm
PL1 Qk	Variable UDL point load 0.500 kN at 800 mm

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PL2 Qk	Variable PL point load 0.900 kN at 600 mm	
Load combinations		
Load combination 1	Support A	Permanent $\times 1.35$
		Variable UDL $\times 1.50$
		Variable PL $\times 0.00$
	Span 1	Permanent $\times 1.35$
		Variable UDL $\times 1.50$
		Variable PL $\times 0.00$
Load combination 2	Support B	Permanent $\times 1.35$
		Variable UDL $\times 1.50$
		Variable PL $\times 0.00$
	Support A	Permanent $\times 1.35$
		Variable UDL $\times 0.00$
		Variable PL $\times 1.50$
Span 1	Permanent $\times 1.35$	
	Variable UDL $\times 0.00$	
	Variable PL $\times 1.50$	
	Support B	Permanent $\times 1.35$
		Variable UDL $\times 0.00$
		Variable PL $\times 1.50$

Analysis results			
Design moment	M = 0.736 kNm	Design shear	F = 1.582 kN
Total load on member	W _{tot} = 3.164 kN		
Reactions at support A	R _{A_max} = 1.582 kN	R _{A_min} = 1.507 kN	
Unfactored permanent load reaction at support A	R _{A_Permanent} = 0.616 kN		
Unfactored variable udl load reaction at support A	R _{A_Variable UDL} = 0.500 kN		
Unfactored variable pl load reaction at support A	R _{A_Variable PL} = 0.450 kN		
Reactions at support B	R _{B_max} = 1.582 kN	R _{B_min} = 1.507 kN	
Unfactored permanent load reaction at support B	R _{B_Permanent} = 0.616 kN		
Unfactored variable udl load reaction at support B	R _{B_Variable UDL} = 0.500 kN		
Unfactored variable pl load reaction at support B	R _{B_Variable PL} = 0.450 kN		

Timber section details			
Breadth of section	b = 47 mm	Depth of section	h = 170 mm
Number of sections	N = 1	Breadth of member	b _b = 47 mm
Inclination of section	θ = 15.0 deg		
Timber strength class	C24		

Member details			
Service class of timber	1	Load duration	Medium-term
Length of span	L _{s1} = 1200 mm		
Length of bearing	L _b = 100 mm		

Compression perpendicular to grain - cl.6.1.4			
Design compressive stress	σ _{c,90,d} = 0.337 N/mm ²	Design compressive strength	f _{c,90,d} = 1.538 N/mm ²
PASS - Design compressive strength exceeds design compressive stress at bearing			

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Biaxial bending - cl 6.1.6

Des.bend.stress in maj.axis $\sigma_{m.y,d} = 3.139 \text{ N/mm}^2$

Des.bend.stress in min.axis $\sigma_{m.z,d} = 3.042 \text{ N/mm}^2$

Design bending strength $f_{m,d} = 14.769 \text{ N/mm}^2$

PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 0.443 \text{ N/mm}^2$

Permissible shear stress $f_{v,d} = 2.462 \text{ N/mm}^2$

PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = 4.800 \text{ mm}$

Total final deflection $\delta_{fin} = 0.808 \text{ mm}$

PASS - Total final deflection is less than the deflection limit

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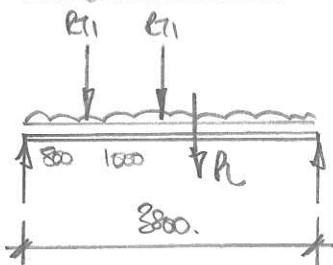
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at 04/12

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Ref Timber R/R Span = 3800mm



$$R_{11} - G_k = 0.12 \text{ kN}$$

$$q_k = 0.5 \text{ kN/m}$$

$$w_{DL} - g_k = 1.05 \times 0.4 = 0.42 \text{ kN/m}$$

$$q_k = 0.75 \times 0.4 = 0.3 \text{ kN/m}$$

Sn13
-15

Use 2 off 50x175 C24 joists R/R

First beam FB1. Span = 2200mm

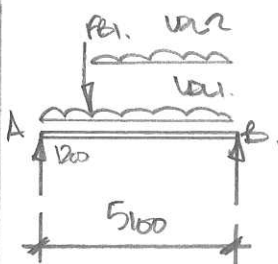
$$\text{loads to beam} - w_{DL} - g_k = 0.73 \times 4 \times 0.5 = 1.5 \text{ kN/m}$$

$$q_k = 1.5 \times 4 \times 0.5 = 3.0 \text{ kN/m}$$

Sn16
+17

Use 2 off 50x175 C24 joists.

First beam FB2 Span = 5100mm



$$F_{B1} - G_k = 2.0 \text{ kN}$$

$$q_k = 4.0 \text{ kN/m}$$

$$w_{DL} - g_k = 0.73 \times 4 \times 0.5 = 1.5 \text{ kN/m}$$

$$q_k = 1.5 \times 4 \times 0.5 = 3.0 \text{ kN/m}$$

$$w_{DL} - g_k = 0.84 \times 6.5 \times 0.5 + 0.33 \times 5.6 \times 0.5 + 1.6 \times 2.4 = 7.5 \text{ kN/m}$$

$$q_k = 0.75 \times 6.5 \times 0.5 + 0.25 \times 5.6 \times 0.5 = 3.1 \text{ kN/m}$$

Sn18
-22

Use 254x146x57 UB beam

$$V_{Ed} = 55.3 \text{ kN}$$

$$V_{Ed} = 56.7 \text{ kN} - \text{use 38mm pads}$$

First beam FB3 Span = 5100mm

$$\text{loads to beam} - w_{DL} - g_k = 1.05 \times 4.2 \times 0.5 + 2.1 \times 2.4 = 7.2 \text{ kN/m}$$

$$q_k = 0.75 \times 4.2 \times 0.5 = 1.6 \text{ kN/m}$$

Sn23
+24

Use 254x146x57 UB beam

$$V_{Ed} = 32.2 \text{ kN}$$

use 225mm bearings/pads

5 Willow Court, Bangor-on-Dee

Extension and Alterations

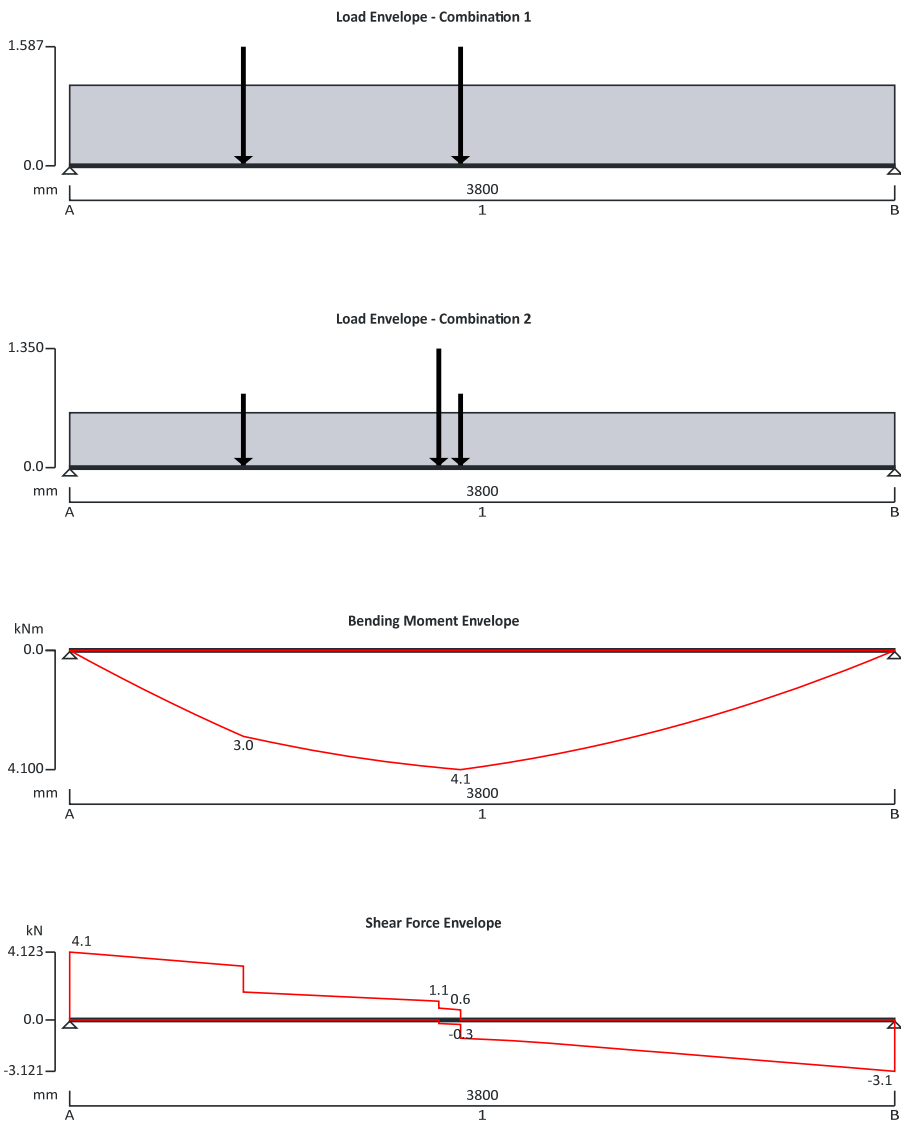
Initials		Date
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Checked by		

ROOF TRIMMER RT2

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.04



Applied loading

Beam loads

RT1 Gk	Permanent self weight of beam × 1
RT1 Qk	Permanent point load 0.620 kN at 800 mm
RT1 Gk	Variable UDL point load 0.500 kN at 800 mm
RT1 Qk	Permanent point load 0.620 kN at 1800 mm
RT1 Qk	Variable UDL point load 0.500 kN at 1800 mm

5 Willow Court, Bangor-on-Dee

Extension and Alterations

Initials

Date

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04/22

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UDL gk

Permanent full UDL 0.400 kN/m

UDL qk

Variable UDL full UDL 0.300 kN/m

PL2 Qk

Variable PL point load 0.900 kN at 1700 mm

Load combinations

Load combination 1

Support A

Permanent $\times$ 1.35Variable UDL $\times$ 1.50Variable PL $\times$ 0.00

Span 1

Permanent $\times$ 1.35Variable UDL $\times$ 1.50Variable PL $\times$ 0.00

Support B

Permanent $\times$ 1.35Variable UDL $\times$ 1.50Variable PL $\times$ 0.00

Load combination 2

Support A

Permanent $\times$ 1.35Variable UDL $\times$ 0.00Variable PL $\times$ 1.50

Span 1

Permanent $\times$ 1.35Variable UDL $\times$ 0.00Variable PL $\times$ 1.50

Support B

Permanent $\times$ 1.35Variable UDL $\times$ 0.00Variable PL $\times$ 1.50**Analysis results**

Design moment

M = **4.100** kNm

Design shear

F = **4.123** kN

Total load on member

W_{tot} = **7.244** kN

Reactions at support A

R_{A_max} = **4.123** kNR_{A_min} = **3.027** kN

Unfactored permanent load reaction at support A

R_{A_Permanent} = **1.690** kN

Unfactored variable udl load reaction at support A

R_{A_Variable UDL} = **1.228** kN

Unfactored variable pl load reaction at support A

R_{A_Variable PL} = **0.497** kN

Reactions at support B

R_{B_max} = **3.121** kNR_{B_min} = **2.357** kN

Unfactored permanent load reaction at support B

R_{B_Permanent} = **1.298** kN

Unfactored variable udl load reaction at support B

R_{B_Variable UDL} = **0.912** kN

Unfactored variable pl load reaction at support B

R_{B_Variable PL} = **0.403** kN**Timber section details**

Breadth of section

b = **50** mm

Depth of section

h = **175** mm

Number of sections

N = **2**

Breadth of member

b_b = **100** mm

Timber strength class

C24**Member details**

Service class of timber

1

Load duration

Medium-term

Length of span

L_{s1} = **3800** mm

Length of bearing

L_b = **100** mm

In accordance with cl.6.6 the member is one of several similar and equally spaced members laterally connected by a continuous load distribution system capable of transferring loads from one member to the neighboring members.

5 Willow Court, Bangor-on-Dee

Extension and Alterations

Initials Date		
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Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c,90,d} = 0.412 \text{ N/mm}^2$ Design compressive strength $f_{c,90,d} = 1.692 \text{ N/mm}^2$
PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m,d} = 8.032 \text{ N/mm}^2$ Design bending strength $f_{m,d} = 16.246 \text{ N/mm}^2$
PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 0.527 \text{ N/mm}^2$ Permissible shear stress $f_{v,d} = 2.708 \text{ N/mm}^2$
PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = 14.000 \text{ mm}$ Total final deflection $\delta_{fin} = 13.793 \text{ mm}$
PASS - Total final deflection is less than the deflection limit

5 Willow Court, Bangor-on-Dee

Extension and Alterations

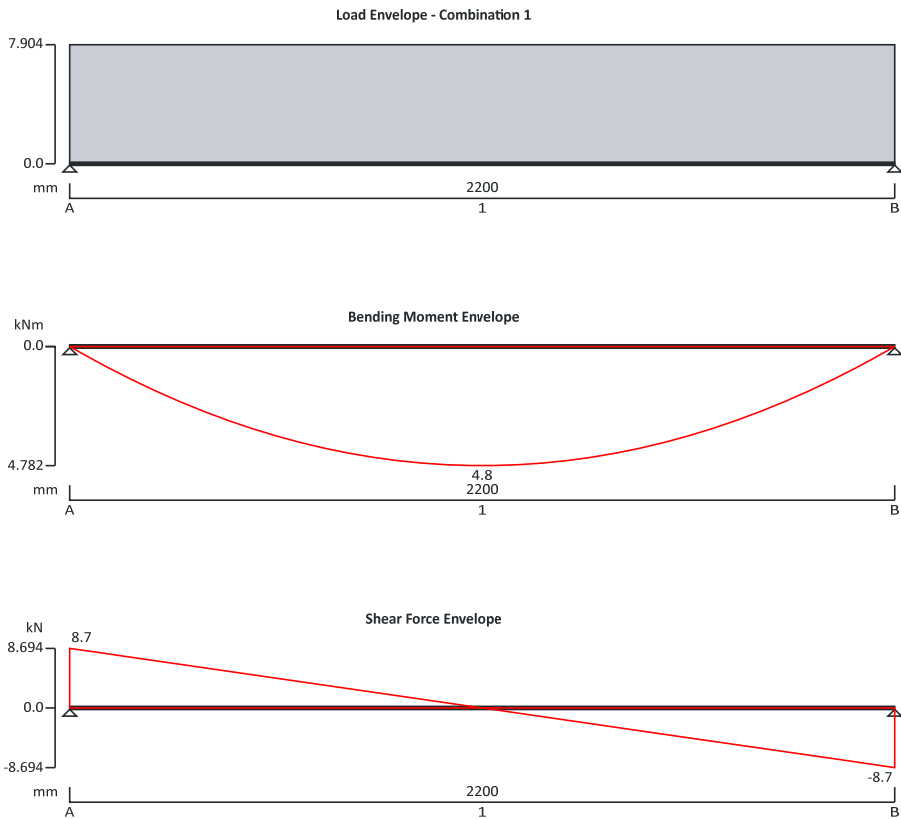
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FLOOR BEAM FB1

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.04



Applied loading

Beam loads

UDL gk	Permanent self weight of beam × 1	
UDL qk	Permanent full UDL 1.800 kN/m	
	Variable full UDL 3.600 kN/m	
Load combinations		
Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50

Analysis results

Design moment	M = 4.782 kNm	Design shear	F = 8.694 kN
---------------	---------------	--------------	--------------

5 Willow Court, Bangor-on-Dee

Extension and Alterations

Initials		Date
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Total load on member $W_{tot} = 17.389$ kN
Reactions at support A $R_{A_max} = 8.694$ kN $R_{A_min} = 8.694$ kN
Unfactored permanent load reaction at support A $R_{A_Permanent} = 2.040$ kN
Unfactored variable load reaction at support A $R_{A_Variable} = 3.960$ kN
Reactions at support B $R_{B_max} = 8.694$ kN $R_{B_min} = 8.694$ kN
Unfactored permanent load reaction at support B $R_{B_Permanent} = 2.040$ kN
Unfactored variable load reaction at support B $R_{B_Variable} = 3.960$ kN

Timber section details

Breadth of section $b = 47$ mm Depth of section $h = 170$ mm
Number of sections $N = 2$ Breadth of member $b_b = 94$ mm
Timber strength class **C24**

Member details

Service class of timber **1** Load duration **Long-term**
Length of span $L_{s1} = 2200$ mm
Length of bearing $L_b = 100$ mm

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c,90,d} = 0.925$ N/mm² Design compressive strength $f_{c,90,d} = 1.346$ N/mm²
PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m,d} = 10.562$ N/mm² Design bending strength $f_{m,d} = 12.923$ N/mm²
PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 1.218$ N/mm² Permissible shear stress $f_{v,d} = 2.154$ N/mm²
PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = 8.800$ mm Total final deflection $\delta_{fin} = 5.674$ mm
PASS - Total final deflection is less than the deflection limit

5 Willow Court, Bangor-on-Dee

Extension and Alterations

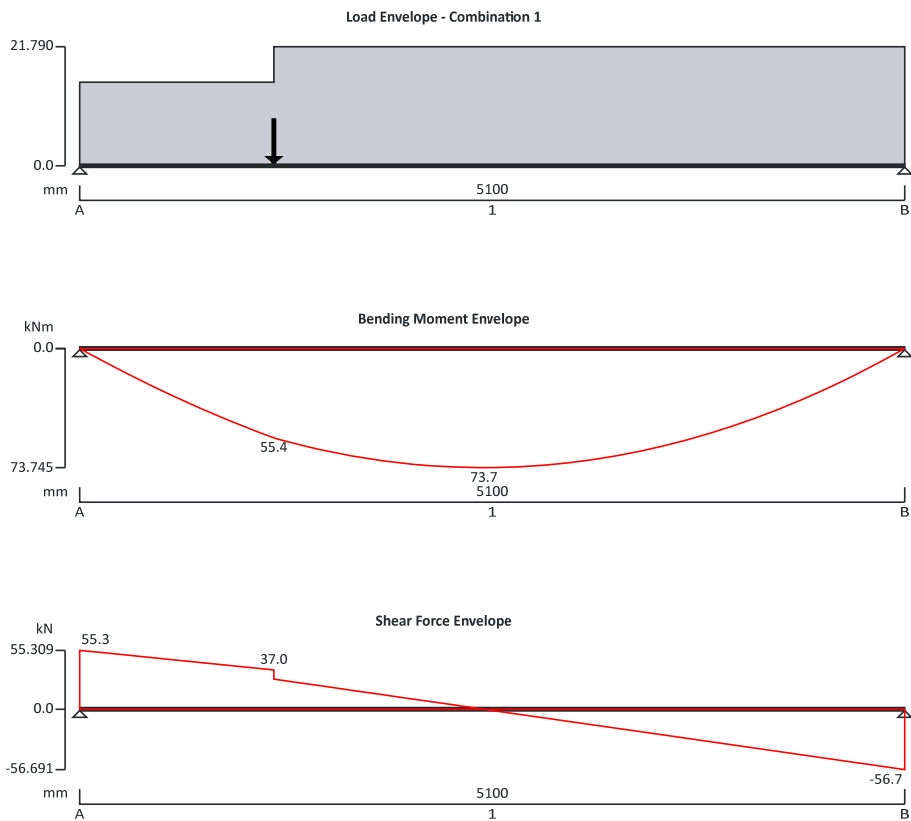
	Initials	Date
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FLOOR BEAM FB2

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam $\times$ 1 FB1 Gk - Permanent point load 2 kN at 1200 mm FB1 Qk - Variable point load 4 kN at 1200 mm UDL1 gk - Permanent full UDL 7.5 kN/m UDL1 qk - Variable full UDL 3.1 kN/m UDL2 gk - Permanent partial UDL 1.5 kN/m from 1200 mm to 5100 mm UDL2 qk - Variable partial UDL 3 kN/m from 1200 mm to 5100 mm
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5 Willow Court, Bangor-on-Dee

Extension and Alterations

Initials

Date

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AH

04/22

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Load combinations

Load combination 1

Support A

Permanent $\times 1.35$ Variable $\times 1.50$ Permanent $\times 1.35$ Variable $\times 1.50$

Support B

Permanent $\times 1.35$ Variable $\times 1.50$ **Analysis results**

Maximum moment

 $M_{\max} = 73.7 \text{ kNm}$ $M_{\min} = 0 \text{ kNm}$

Maximum shear

 $V_{\max} = 55.3 \text{ kN}$ $V_{\min} = -56.7 \text{ kN}$

Deflection

 $\delta_{\max} = 12.2 \text{ mm}$ $\delta_{\min} = 0 \text{ mm}$

Maximum reaction at support A

 $R_{A_{\max}} = 55.3 \text{ kN}$ $R_{A_{\min}} = 55.3 \text{ kN}$

Unfactored permanent load reaction at support A

 $R_{A_{\text{Permanent}}} = 23.8 \text{ kN}$

Unfactored variable load reaction at support A

 $R_{A_{\text{Variable}}} = 15.4 \text{ kN}$

Maximum reaction at support B

 $R_{B_{\max}} = 56.7 \text{ kN}$ $R_{B_{\min}} = 56.7 \text{ kN}$

Unfactored permanent load reaction at support B

 $R_{B_{\text{Permanent}}} = 24.1 \text{ kN}$

Unfactored variable load reaction at support B

 $R_{B_{\text{Variable}}} = 16.1 \text{ kN}$ **Section details**

Section type

UKB 254x146x37 (Tata Steel Advance)

Steel grade

S355

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element

 $t = \max(t_f, t_w) = 10.9 \text{ mm}$

Nominal yield strength

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

Nominal ultimate tensile strength

 $f_u = 470 \text{ N/mm}^2$

Modulus of elasticity

 $E = 210000 \text{ N/mm}^2$ **Partial factors - Section 6.1**

Resistance of cross-sections

 $\gamma_{M0} = 1.00$

Resistance of members to instability

 $\gamma_{M1} = 1.00$

Resistance of tensile members to fracture

 $\gamma_{M2} = 1.10$ **Lateral restraint**

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis

 $K_y = 1.000$

Effective length factor in minor axis

 $K_z = 1.000$

Effective length factor for torsion

 $K_{LT,A} = 1.000$ $K_{LT,B} = 1.000$ **Classification of cross sections - Section 5.5** $\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$ **Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)**

Width of section

 $c = d = 219 \text{ mm}$ $c / t_w = 42.7 \times \varepsilon \leq 72 \times \varepsilon$

Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

 $c = (b - t_w - 2 \times r) / 2 = 62.5 \text{ mm}$ $c / t_f = 7.0 \times \varepsilon \leq 9 \times \varepsilon$

Class 1

5 Willow Court, Bangor-on-Dee

Extension and Alterations

Initials Date

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Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 234.2 \text{ mm}$$

Shear area factor

$$\eta = 1.000$$

$$h_w / t_w < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 56.7 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1759 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 360.6 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force**Check bending moment major (y-y) axis - Section 6.2.5**

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 73.7 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 171.5 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.94$$

$$C_1 = 1 / k_c^2 = 1.132$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = 0.947$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$$

Unrestrained length

$$L = 1.0 \times L_{s1} = 5100 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 111.7 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.239$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

 $\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored**Design resistance for buckling - Section 6.3.2.1**

Buckling curve - Table 6.5

$$b$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = 0.34$$

Correction factor for rolled sections

$$\beta = 0.75$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.219$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1 / \bar{\lambda}_{LT}^2) = 0.557$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.982$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.567$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 97.3 \text{ kNm}$$

PASS - Design buckling resistance moment exceeds design bending moment**Check vertical deflection - Section 7.2.1**

Consider deflection due to permanent and variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = 14.2 \text{ mm}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 12.211 \text{ mm}$$

PASS - Maximum deflection does not exceed deflection limit

5 Willow Court, Bangor-on-Dee

Extension and Alterations

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FLOOR BEAM RB3 - BEARING (B)

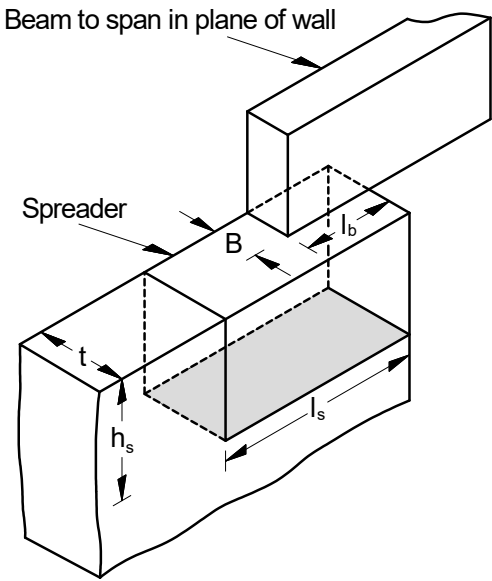
MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.08

Masonry details

Masonry type
Compressive strength of unit
Mortar designation
Least horizontal dimension of masonry units
Height of masonry units
Category of masonry units
Category of construction control
Partial safety factor for material strength
Thickness of load bearing leaf
Effective thickness of masonry wall
Height of masonry wall
Effective height of masonry wall

Aggregate concrete blocks (25% or less formed voids)
 $p_{unit} = 3.6 \text{ N/mm}^2$
iii
 $l_{unit} = 100 \text{ mm}$
 $h_{unit} = 215 \text{ mm}$
Category I
Normal
 $\gamma_m = 3.1$
 $t = 100 \text{ mm}$
 $t_{ef} = 135 \text{ mm}$
 $h = 2400 \text{ mm}$
 $h_{ef} = 2400 \text{ mm}$



Bearing details

Beam spanning in plane of wall
Width of bearing
Length of bearing

$B = 100 \text{ mm}$
 $l_b = 325 \text{ mm}$

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

Mortar designation
Block compressive strength
Characteristic compressive strength (Table 2c)
Characteristic compressive strength (Table 2d)
Height of solid block

Mortar = "iii"
 $p_{unit} = 3.6 \text{ N/mm}^2$
 $f_{kc} = 1.70 \text{ N/mm}^2$
 $f_{kd} = 3.50 \text{ N/mm}^2$
 $h_{unit} = 215.0 \text{ mm}$

5 Willow Court, Bangor-on-Dee

Extension and Alterations

Initials

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Least horizontal dimension

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

Block ratio

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

Ratio between 0.6 and 4.5 - OK

Characteristic compressive strength

 $f_k = 3.50 \text{ N/mm}^2$

Loading details

Characteristic concentrated dead load

 $G_k = 24 \text{ kN}$

Characteristic concentrated imposed load

 $Q_k = 16 \text{ kN}$

Design concentrated load

 $F = (G_k \times 1.4) + (Q_k \times 1.6) = 59.5 \text{ kN}$

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

Not applicable

Bearing safety factor

 $\gamma_{bear} = 1.00$

Check design bearing without a spreader

Design bearing stress

 $f_{ca} = F / (B \times l_b) + f / t = 1.831 \text{ N/mm}^2$

Allowable bearing stress

 $f_{cp} = \gamma_{bear} \times f_k / \gamma_m = 1.129 \text{ N/mm}^2$

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

Spreader details

Length of spreader

 $l_s = 330 \text{ mm}$

Depth of spreader

 $h_s = 215 \text{ mm}$

Edge distance

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

Spreader bearing type

Bearing type

Type 3

Bearing safety factor

 $\gamma_{bear} = 2.00$

Check design bearing with a spreader

Loading acts eccentrically within middle third – triangular stress distribution

Eccentricity of load

 $e = (l_s - l_b) / 2 = 3 \text{ mm}$

Maximum bearing stress

 $f_{ca} = F \times (1 + (6 \times e / l_s)) / (l_s \times t) + f / t = 1.885 \text{ N/mm}^2$

Allowable bearing stress

 $f_{cp} = \gamma_{bear} \times f_k / \gamma_m = 2.258 \text{ N/mm}^2$

PASS - Allowable bearing stress exceeds design bearing stress

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

Slenderness ratio

 $h_{ef} / t_{ef} = 17.78$

Eccentricity at top of wall

 $e_x = 0.0 \text{ mm}$

From BS5628:1 Table 7

Capacity reduction factor

 $\beta = 0.84$

Length of bearing distributed at $0.4 \times h$
 $l_d = 1285 \text{ mm}$

Maximum bearing stress

 $f_{ca} = F / (l_d \times t) + f / t = 0.463 \text{ N/mm}^2$

Allowable bearing stress

 $f_{cp} = \beta \times f_k / \gamma_m = 0.952 \text{ N/mm}^2$

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

5 Willow Court, Bangor-on-Dee

Extension and Alterations

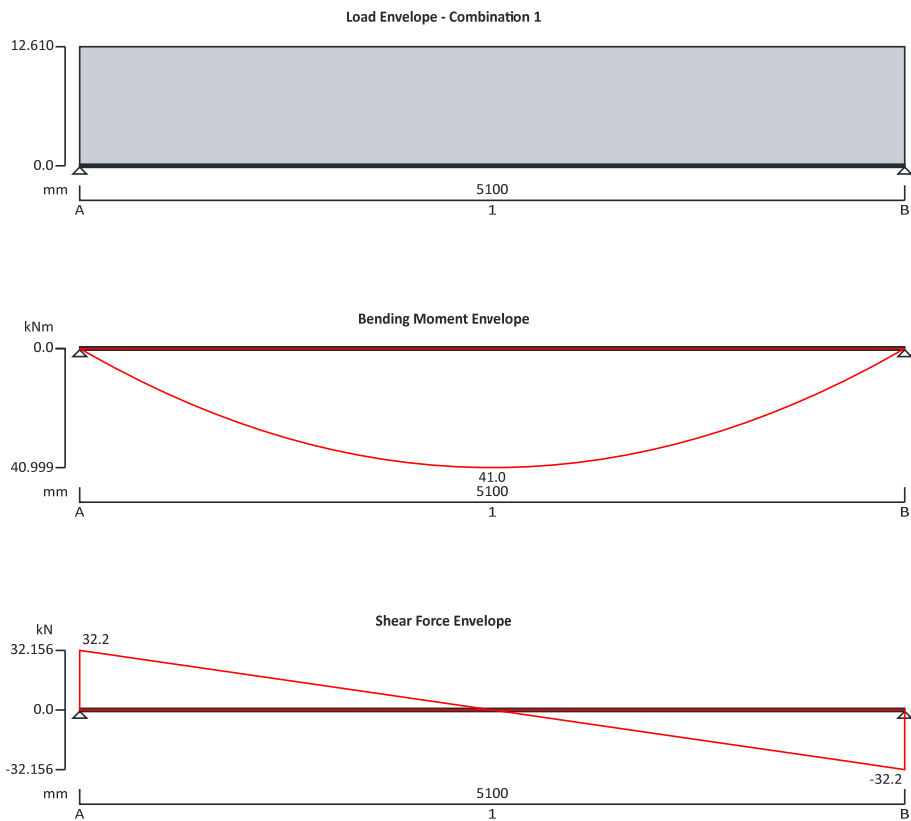
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FLOOR BEAM FB3

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam $\times 1$
	UDL gk - Permanent full UDL 7.2 kN/m
	UDL qk - Variable full UDL 1.6 kN/m

Load combinations

Load combination 1	Support A	Permanent $\times 1.35$
		Variable $\times 1.50$
		Permanent $\times 1.35$
		Variable $\times 1.50$
	Support B	Permanent $\times 1.35$

5 Willow Court, Bangor-on-Dee

Extension and Alterations

Initials		Date
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Variable $\times 1.50$

Analysis results

Maximum moment	$M_{max} = 41 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 32.2 \text{ kN}$	$V_{min} = -32.2 \text{ kN}$
Deflection	$\delta_{max} = 6.9 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 32.2 \text{ kN}$	$R_{A_min} = 32.2 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 19.3 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 4.1 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 32.2 \text{ kN}$	$R_{B_min} = 32.2 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 19.3 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 4.1 \text{ kN}$	

Section details

Section type	UKB 254x146x37 (Tata Steel Advance)	Steel grade	S355
Section classification	Class 1		

Check shear - Section 6.2.6

Design shear force	$V_{Ed} = 32 \text{ kN}$	Design shear resistance	$V_{c,Rd} = 360.6 \text{ kN}$
PASS - Design shear resistance exceeds design shear force			

Check bending moment - Section 6.2.5

Design bending moment	$M_{Ed} = 41 \text{ kNm}$	Des.bending resist.moment	$M_{c,Rd} = 171.5 \text{ kNm}$
-----------------------	---------------------------	---------------------------	--------------------------------

Slenderness ratio for lateral torsional buckling

LTB slenderness ratio	$\bar{\lambda}_{LT} = 1.239$	Limiting slenderness ratio	$\bar{\lambda}_{LT,0} = 0.400$
$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored			

Design resistance for buckling - Section 6.3.2.1

Des.buckling resist.moment	$M_{b,Rd} = 97.3 \text{ kNm}$	PASS - Design buckling resistance moment exceeds design bending moment	
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Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads			
Limiting deflection	$\delta_{lim} = 14.2 \text{ mm}$	Maximum deflection	$\delta = 6.942 \text{ mm}$
PASS - Maximum deflection does not exceed deflection limit			

SCHEME

PAGE No.

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5 Willow Court, Bangor-on-Dee

Initials Date

Extension and Alterations

Prepared by

Aut 04/12

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Unit L1 Eff Span = 2100mm.

$$\begin{aligned} \text{loads to Unit} - G_k &= 2.1 \times 2.1^2 / 4 + 1.05 \times 4.2 \times 2.1 = 6.7 \text{ kJ} \\ Q_k &= 0.75 \times 4.2 \times 2.1 = 3.2 \text{ kJ} \end{aligned} \quad \left. \vphantom{\begin{aligned} \text{loads to Unit} - G_k \\ Q_k \end{aligned}} \right\} 10.9 \text{ kJ}.$$

Use Kystrae Box/c-100 Unit S.W. = 30 kJ or

Unit L2 Eff Span = 2400mm.

$$\begin{aligned} \text{loads to Unit} - W_k - G_k &= 1.05 \times 4.2 = 2.2 \text{ kJ/m} \\ Q_k &= 0.75 \times 4.2 = 1.6 \text{ kJ/m} \end{aligned}$$

Use 2ft 50x175 C16 joists □.

Unit L3 Eff Span = 1800mm

$$\text{Masonry with arch over opening} - G_k = (1.8 + 1.6) \times 1.8^2 / 4 = 2.8 \text{ kJ}.$$

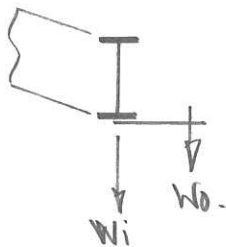
Use Kystrae S/c-90 Unit S.W. = 19 kJ or

Unit L4 Eff Span = 4200mm.

$$W_i - G_k = 1.05 \times 4.2 = 2.2 \text{ kJ/m}$$

$$Q_k = 0.75 \times 4.2 = 1.6 \text{ kJ/m}$$

$$W_o - \text{doors} - G_k = 0.7 \times 2.1 = 1.5 \text{ kJ/m}.$$



Use 203x133x30 UB + 275x8 plate $V_{Ed} = 17.3 \text{ kJ}$

Post P1 Ht = 2700mm say

Vertical load from L4 - $N_{Ed} = 17.3 \text{ kJ}$.

$$\text{width of flange required} = 7/2 = 3.5 \text{ m} \quad \therefore W_k = 1.0 \times 3.5 = 3.5 \text{ kJ/m nt.}$$

Use 100x100x5.0 SxS posts.

SOLID WALL - BOX LINTELS

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WALL WIDTHS
100mm-215mm

BOX/K-100

Used to support openings in 100mm wide walls.
Lintels may be propped to facilitate speed of construction. See Installation on page 6.



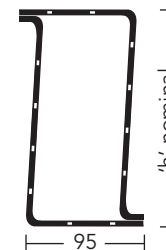
Standard Load

BOX/K-100

Manufactured length 150mm increments	600- 1500	1650- 2100	2250- 2400	2550- 2700	2850- 3600	3750- 4200	4350- 4800
Height 'h'	75	150	150	150	215	215	215
Thickness	1.6	2.0	2.0	2.0	2.5	2.5	2.5
Total UDL kN	15	30	25	20	30	25	20

Used to support openings in 100mm wide walls.

Standard Load



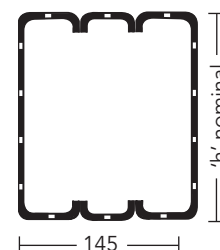
45

BOX/K-150

Manufactured length 150mm increments	600- 1500	1650- 2100	2250- 2400	2550- 2700	2850- 3600	3750- 4200	4350- 4800
Height 'h'	150	150	150	150	215	215	215
Thickness	1.6	2.0	2.0	2.0	2.5	2.5	2.5
Total UDL kN	15	30	25	20	35	30	25

Used to support openings in 150mm wide walls.

Standard Load



BOX/K-200

Manufactured length 150mm increments	600- 1500	1650- 2100	2250- 2400	2550- 2700	2850- 3600	3750- 4200	4350- 4800
Height 'h'	150	150	150	150	215	215	215
Thickness	1.6	2.0	2.0	2.0	2.5	2.5	2.5
Total UDL kN	15	30	25	20	30	25	20

The flange of the BOX/K-200 is designed to support a nominal masonry load only up to a maximum of 3kN per metre run.
Used to support openings in 215mm wide walls.

Standard Load

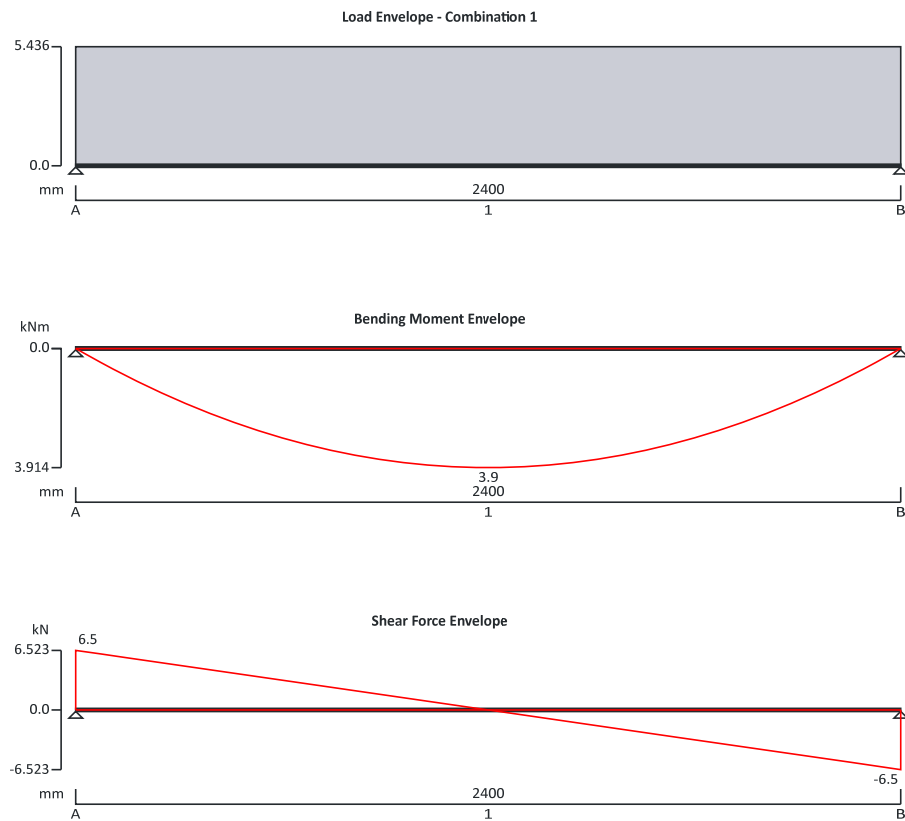


LINTEL L2

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.04



Applied loading

Beam loads

	Permanent self weight of beam $\times 1$	
UDL gk	Permanent full UDL 2.200 kN/m	
UDL qk	Variable full UDL 1.600 kN/m	
Load combinations		
Load combination 1	Support A	Permanent $\times 1.35$
		Variable $\times 1.50$
	Span 1	Permanent $\times 1.35$
		Variable $\times 1.50$
	Support B	Permanent $\times 1.35$
		Variable $\times 1.50$

Analysis results

Design moment	M = 3.914 kNm	Design shear	F = 6.523 kN
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5 Willow Court, Bangor-on-Dee

Extension and Alterations

Initials		Date
Prepared by	AH	04/22
Checked by		

Total load on member $W_{tot} = 13.045 \text{ kN}$
Reactions at support A $R_{A_max} = 6.523 \text{ kN}$ $R_{A_min} = 6.523 \text{ kN}$
Unfactored permanent load reaction at support A $R_{A_Permanent} = 2.698 \text{ kN}$
Unfactored variable load reaction at support A $R_{A_Variable} = 1.920 \text{ kN}$
Reactions at support B $R_{B_max} = 6.523 \text{ kN}$ $R_{B_min} = 6.523 \text{ kN}$
Unfactored permanent load reaction at support B $R_{B_Permanent} = 2.698 \text{ kN}$
Unfactored variable load reaction at support B $R_{B_Variable} = 1.920 \text{ kN}$

Timber section details

Breadth of section $b = 47 \text{ mm}$ Depth of section $h = 170 \text{ mm}$
Number of sections $N = 2$ Breadth of member $b_b = 94 \text{ mm}$
Timber strength class **C16**

Member details

Service class of timber **1** Load duration **Medium-term**
Length of span $L_{s1} = 2400 \text{ mm}$
Length of bearing $L_b = 100 \text{ mm}$

Compression perpendicular to grain - cl.6.1.4

Design compressive stress $\sigma_{c,90,d} = 0.694 \text{ N/mm}^2$ Design compressive strength $f_{c,90,d} = 1.354 \text{ N/mm}^2$
PASS - Design compressive strength exceeds design compressive stress at bearing

Bending - cl 6.1.6

Design bending stress $\sigma_{m,d} = 8.644 \text{ N/mm}^2$ Design bending strength $f_{m,d} = 9.846 \text{ N/mm}^2$
PASS - Design bending strength exceeds design bending stress

Shear - cl.6.1.7

Applied shear stress $\tau_d = 0.914 \text{ N/mm}^2$ Permissible shear stress $f_{v,d} = 1.969 \text{ N/mm}^2$
PASS - Design shear strength exceeds design shear stress

Deflection - cl.7.2

Deflection limit $\delta_{lim} = 9.600 \text{ mm}$ Total final deflection $\delta_{fin} = 8.290 \text{ mm}$
PASS - Total final deflection is less than the deflection limit

CAVITY WALL

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Cavity widths from 50mm-165mm

OUTER LEAF
102mm

INNER LEAF
100mm

S/K

Blockwork built tight against inner face of the lintel.
Place mortar bed on top of blockwork before floor units are laid to provide even distribution of load.
Lintels may be propped to facilitate speed of construction. See Lintel Installation on page 6.

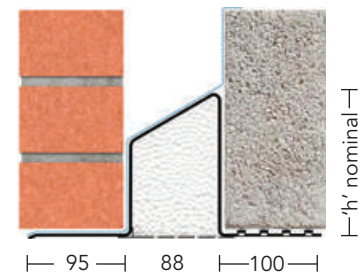


Standard Load

S/K-90 (For cavity widths 90-105mm)

Manufactured length 150mm increments	600- 1200	1350- 1500	1650- 1800	1950- 2100	2250- 2400	2550- 2700	2850- 3000	3150- 3600	3750- 4000	4200- 4800
Height 'h'	88	85	107	125	150	162	171	200	200	200
Thickness	1.6	2.0	2.0	2.0	2.0	2.6	2.6	3.2	3.2	3.4
Total UDL kN 3:1	12	16	19	21	23	27	27	27	26	27
Total UDL kN 19:1	10	13	14	17	18	22	20	20	19	22

90-105mm Cavity

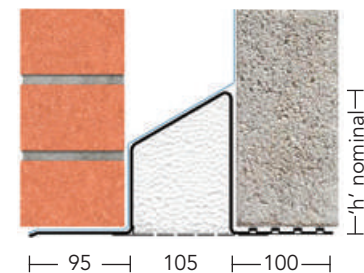


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S/K-110 (For cavity widths 110-125mm)

Manufactured length 150mm increments	600- 1500	1650- 1800	1950- 2100	2250- 2700	2850- 3000	3150- 4000	4200- 4800
Height 'h'	97	113	125	163	193	193	218
Thickness	2.0	2.0	2.0	2.5	2.9	3.2	3.2
Total UDL kN 3:1	16	22	21	25	27	26	25
Total UDL kN 19:1	13	18	17	20	22	19	20

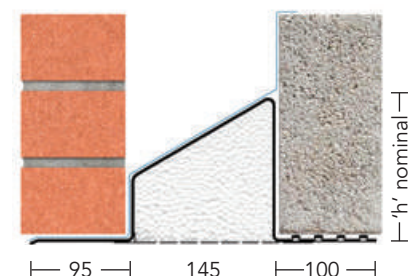
110-125mm Cavity



S/K-150 (For cavity widths 150-165mm)

Manufactured length 150mm increments	600- 1500	1650- 1800	1950- 2100	2250- 2700	2850- 3000	3150- 4000	4200- 4800
Height 'h'	100	118	130	148	173	173	199
Thickness	2.0	2.0	2.0	2.5	2.9	3.2	3.2
Total UDL kN 3:1	16	22	21	25	27	26	25
Total UDL kN 19:1	13	18	17	20	22	19	20

150-165mm Cavity



5 Willow Court, Bangor-on-Dee

Extension and Alterations

Initials Date		
Prepared by	AH	04/22
Checked by		

LINTEL L4

STEEL MASONRY SUPPORT

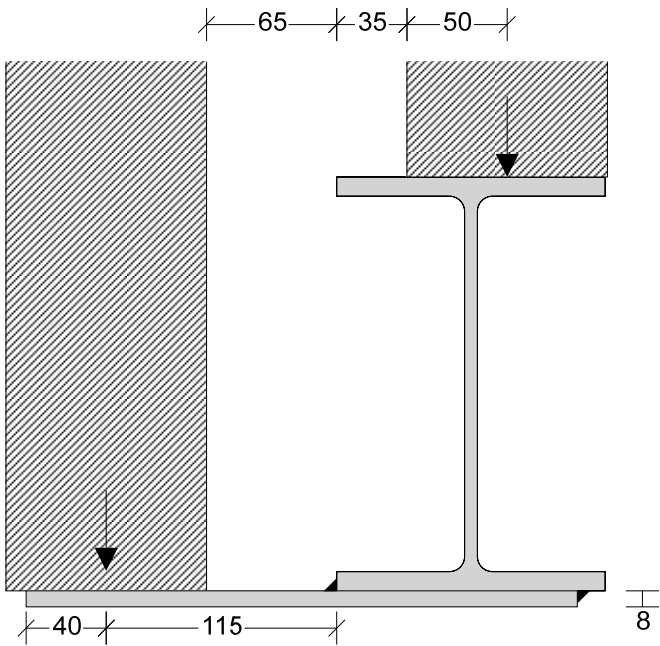
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Tedds calculation version 1.0.02

Design summary

Overall design status PASS
Overall design utilisation 0.925

Description	Unit	Allowable	Applied	Utilisation	Result
Heel moment	kNm/m	2.933	0.247	0.084	PASS
Deflection	mm	1.9	0.1	0.072	PASS
Weld capacity	kN/m	945.2	290.3	0.307	PASS
Shear force (major axis)	kN	215.5	17.3	0.080	PASS
Bending (major-axis)	kNm	58.3	18.2	0.312	PASS
Bending (minor axis)	kNm	24.3	1.2	0.049	PASS
Warping	kNm	11.8	1.4	0.115	PASS
Bending and torsion				0.464	PASS
Plastic interaction				0.208	PASS
Torsion beam rotation	deg	3.00	2.77	0.925	PASS
Torsion beam deflection	mm	10.0	4.0	0.395	PASS



Partial factors - Section 6.1

Resistance of cross-sections $\gamma_{M0} = 1$
Resist. of members to instability $\gamma_{M1} = 1$
Resistance of joints $\gamma_{M2} = 1.25$
Partial factor for permanent action $\gamma_G = 1.35$
Partial factor for variable action $\gamma_Q = 1.50$
Partial factor for permanent action (favourable) $\gamma_{G_fav} = 1.00$
Partial factor for variable action (favourable) $\gamma_{Q_fav} = 0.00$

5 Willow Court, Bangor-on-Dee

Extension and Alterations

Initials Date

Prepared by

AH

04/22

Checked by

Steel beam section details

Torsion beam section type

UB 203x133x30

Nominal yield strength

 $f_y = f_{y,tb} = 275 \text{ N/mm}^2$

Nominal ultimate tensile strength

 $f_u = f_{u,tb} = 410 \text{ N/mm}^2$ **Masonry support section details**

Section type

Plate 275x8(155)

Steel grade - EN 10025-2:2004

S275

Nominal thickness

 $t_{nom,sb} = t_{plate} = 8 \text{ mm}$

Nominal yield strength

 $f_{y,sb} = 275 \text{ N/mm}^2$

Nominal ultimate tensile strength

 $f_{u,sb} = 410 \text{ N/mm}^2$

Modulus of elasticity

 $E_{sb} = 210000 \text{ N/mm}^2$

Total length of plate

 $l_{plate} = 275 \text{ mm}$

Length of plate beyond outer edge of torsion beam

 $l_h = 155 \text{ mm}$ **Supported materials detail**

Density of masonry on torsion beam

 $\rho_{m,tb} = 16.0 \text{ kN/m}^3$

Width of masonry on torsion beam

 $b_{m,tb} = 100 \text{ mm}$

Height of masonry on torsion beam

 $h_{m,tb} = 50 \text{ mm}$

Eccentricity of torsion beam masonry

 $e_{load,tb} = 85 \text{ mm}$

Eccentricity of torsion beam material

 $e_{tb} = 35 \text{ mm}$

Add perm. force torsion beam (not masonry)

 $G_{k,add,tb} = 2.2 \text{ kN/m}$

Add var. force torsion beam (not masonry)

 $Q_{k,add,tb} = 1.6 \text{ kN/m}$

Density of masonry on support beam

 $\rho_{m,sb} = 18.0 \text{ kN/m}^3$

Width of masonry on support beam

 $b_{m,sb} = 100 \text{ mm}$

Height of masonry on support beam

 $h_{m,sb} = 50 \text{ mm}$

Eccentricity of support beam masonry

 $e_{load,sb} = 115 \text{ mm}$

Add perm. force support beam (not masonry)

 $G_{k,add,sb} = 1.5 \text{ kN/m}$

Add var. force support beam (not masonry)

 $Q_{k,add,sb} = 0.0 \text{ kN/m}$ **Geometry**

Cavity width

 $b_{cavity} = 100 \text{ mm}$

Supported width of masonry

 $d_m = l_h + t_{shim} + e_{tb} - b_{cavity} = 90 \text{ mm}$ **Biaxial stress effects in the plate (SCI-P-110)**

Maximum overall bending moment

 $M_{y,Ed} = 18.2 \text{ kNm}$

Dist to NA combined section (CoG torsion beam)

 $Z_{na,all} = (h_{tb} + t_{plate}) \times A_{pl} / (2 \times (A_{tb} + A_{pl})) = 39 \text{ mm}$

Second moment of area of combined section

 $I_{y,all} = (I_{y,tb} + A_{tb} \times Z_{na,all}^2) + A_{pl} \times (h_{tb} / 2 + t_{plate} / 2 - Z_{na,all})^2 = 4506 \text{ cm}^4$

Elastic section modulus of combined section

 $Z_{y,all} = I_{y,all} / (h_{tb} / 2 + t_{plate} - Z_{na,all}) = 624.46 \text{ cm}^3$

Section modulus of plate

 $Z_{y,plate} = 1m \times t_{plate}^2 / (6 \times 1m) = 10.67 \text{ cm}^3/m$

Force of masonry on support plate

 $F_1 = (b_{m,sb} \times h_{m,sb} \times \rho_{m,sb} + G_{k,add,sb}) \times \gamma_G + Q_{k,add,sb} \times \gamma_Q = 2.1 \text{ kN/m}$

Bending at heel

 $M_{y,Ed,plate} = F_1 \times e_{load,sb} = 0.2 \text{ kNm/m}$

Moment capacity of plate

 $M_{y,Rd,plate} = Z_{y,plate} \times f_{y,sb} / \gamma_{M0} = 2.9 \text{ kNm/m}$ **PASS - Moment capacity of plate exceeds applied moment**

Longitudinal stress due to overall bending

 $\sigma_1 = M_{y,Ed} / Z_{y,all} = 29.1 \text{ N/mm}^2$

Constant relating to Von Mises curve

 $C_{fp} = (4 \times f_{y,sb}^2 - 3 \times \sigma_1^2)^{0.5} = 547.7 \text{ N/mm}^2$

Transverse bending stress ratio limit

 $\alpha_{ts} = (C_{fp}^2 - \sigma_1^2) / (2 \times C_{fp} \times f_{y,sb}) = 0.993$

Transverse bending stress ratio

 $\alpha_{ls} = M_{y,Ed,plate} / M_{y,Rd,plate} = 0.084$ **PASS - Transverse bending stress ratio less than allowable limit**

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Extension and Alterations

Initials Date

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04/22

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Deflection of plate

Unfactored force on support angle

$$F_{1ser} = (b_{m, sb} \times h_{m, sb} \times \rho_{m, sb} + G_{k, add, sb}) + Q_{k, add, sb} = 1.6 \text{ kN/m}$$

Distance from weld to load position

$$a_m = e_{load, sb} = 115 \text{ mm}$$

Length of load resultant to edge of plate

$$b_m = l_h - e_{load, sb} = 40 \text{ mm}$$

Dist from weld to load position as ratio of length

$$a_l = a_m / (a_m + b_m) = 0.742$$

Effective second moment of area

$$I_{eff_def} = t_{plate}^3 / 12 = 42667 \text{ mm}^4/\text{m}$$

Deflection at end of plate

$$\delta = (a_l^2 \times (3 - a_l) / 6) \times (F_{1ser} \times (a_m + b_m)^3) / (E_{sb} \times I_{eff_def}) = 0.14 \text{ mm}$$

Deflection limit

$$\delta_{lim} = \min((1 + d_m / b_{cavity}) \times 1\text{mm}, 2\text{mm}) = 1.90 \text{ mm}$$

PASS - Deflection is within specified criteria**Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam**

Shear force at weld position

$$F_A = F_1 \times \max((1 + (3 \times e_{load, sb}) / (2 \times b_{tb} / 2)), 1.4) = 7.7 \text{ kN/m}$$

Maximum possible force in plate

$$F_p = (l_h + \min(b_{tb}, l_{plate} - l_h)) \times t_{plate} \times f_{y, sb} = 605.0 \text{ kN}$$

Longitudinal shear between beam and plate

$$F_l = 2 \times F_p / L = 288.1 \text{ kN/m}$$

Horizontal shear between beam and plate

$$F_h = F_1 \times e_{load, sb} / (s_{weld} / 2 + t_{plate} / 2) = 35.3 \text{ kN/m}$$

Resultant weld force

$$F_{w, Ed} = (F_A^2 + F_l^2 + F_h^2)^{0.5} = 290.3 \text{ kN/m}$$

Leg length of weld

$$s_{weld} = 6.00 \text{ mm}$$

Throat thickness of weld

$$a_{weld} = 1 / \sqrt{2} \times s_{weld} = 4.24 \text{ mm}$$

Length of weld per metre run

$$l_{weld} = 1000 \text{ mm/m}$$

Ultimate tensile strength used for weld

$$f_{u, weld} = \min(f_{u, sb}, f_{u, tb}) = 410.0 \text{ N/mm}^2$$

Correlation factor (table 4.1)

$$\beta_w = 0.85$$

Design shear strength

$$f_{vw, d} = f_{u, weld} / (\sqrt{3} \times \beta_w \times \gamma_{M2}) = 222.8 \text{ N/mm}^2$$

Design resistance of weld

$$F_{w, Rd} = f_{vw, d} \times a_{weld} = 945.2 \text{ kN/m}$$

PASS - weld capacity exceeds applied force**Eccentricities**

Distance to shear centre of torsion beam

$$e_{0, tb} = 0 \text{ mm}$$

Eccentricity of support beam masonry

$$e_{m, sb} = e_{load, sb} + b_{tb} / 2 = 182 \text{ mm}$$

Eccentricity of torsion beam masonry

$$e_{m, tb} = b_{tb} / 2 - e_{load, tb} = -18 \text{ mm}$$

Eccentricity of support beam

$$e_{b, sb} = c_{zsb} + b_{tb} / 2 = 84 \text{ mm}$$

Eccentricity of torsion beam

$$e_{b, tb} = 0 \text{ mm}$$

Torsional loading ULS (unfavourable)

Loading of support beam masonry

$$w_{sb} = (h_{m, sb} \times b_{m, sb} \times \rho_{m, sb} + G_{k, add, sb}) \times \gamma_G = 2.15 \text{ kN/m}$$

Loading of torsion beam masonry

$$w_{tb} = (h_{m, tb} \times b_{m, tb} \times \rho_{m, tb} + G_{k, add, tb}) \times \gamma_G + Q_{k, add, tb} \times \gamma_Q = 5.48 \text{ kN/m}$$

Self weight of support beam

$$w_{sw, sb} = A_{pl} \times \rho_{SEC3} \times g_{acc} \times \gamma_G = 0.23 \text{ kN/m}$$

Self weight of torsion beam

$$w_{sw, tb} = A_{tb} \times \rho_{SEC3} \times g_{acc} \times \gamma_G = 0.40 \text{ kN/m}$$

Torsional loading ULS (favourable)

Loading of support beam masonry

$$w_{sb_fav} = (h_{m, sb} \times b_{m, sb} \times \rho_{m, sb} + G_{k, add, sb}) \times \gamma_{G_fav} = 1.59 \text{ kN/m}$$

Loading of torsion beam masonry

$$w_{tb_fav} = (h_{m, tb} \times b_{m, tb} \times \rho_{m, tb} + G_{k, add, tb}) \times \gamma_{G_fav} + Q_{k, add, tb} \times \gamma_{Q_fav} = 2.28 \text{ kN/m}$$

Self weight of support beam

$$w_{sw, sb_fav} = A_{pl} \times \rho_{SEC3} \times g_{acc} \times \gamma_{G_fav} = 0.17 \text{ kN/m}$$

Self weight of torsion beam

$$w_{sw, tb_fav} = A_{tb} \times \rho_{SEC3} \times g_{acc} \times \gamma_{G_fav} = 0.29 \text{ kN/m}$$

Torsional loading SLS (unfavourable)

Loading of support beam masonry

$$w_{sb, ser} = h_{m, sb} \times b_{m, sb} \times \rho_{m, sb} + G_{k, add, sb} = 1.59 \text{ kN/m}$$

Loading of torsion beam masonry

$$w_{tb, ser} = h_{m, tb} \times b_{m, tb} \times \rho_{m, tb} + G_{k, add, tb} + Q_{k, add, tb} = 3.88 \text{ kN/m}$$

5 Willow Court, Bangor-on-Dee

Extension and Alterations

Initials Date

Prepared by

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04/22

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Self weight of support beam

$$W_{sw, sb, ser} = A_{pl} \times \rho_{SEC3} \times g_{acc} = \mathbf{0.17 \text{ kN/m}}$$

Self weight of torsion beam

$$W_{sw, tb, ser} = A_{tb} \times \rho_{SEC3} \times g_{acc} = \mathbf{0.29 \text{ kN/m}}$$

Torsional effects

Applied torque (ULS +ve ecc. unfav, -ve ecc. fav)

$$T_{d, w, fav1} = \text{abs}(W_{sb} \times e_{m, sb} + W_{tb, fav} \times e_{m, tb} + W_{sw, sb} \times e_{b, sb} + W_{sw, tb} \times e_{b, tb}) = \mathbf{0.37 \text{ kNm/m}}$$

Applied torque (ULS +ve ecc. fav, -ve ecc. unfav)

$$T_{d, w, fav2} = \text{abs}(W_{sb, fav} \times e_{m, sb} + W_{tb} \times e_{m, tb} + W_{sw, sb, fav} \times e_{b, sb} + W_{sw, tb, fav} \times e_{b, tb}) = \mathbf{0.20 \text{ kNm/m}}$$

Applied torque (ULS all unfavourable)

$$T_{d, w} = \text{abs}(W_{sb} \times e_{m, sb} + W_{tb} \times e_{m, tb} + W_{sw, sb} \times e_{b, sb} + W_{sw, tb} \times e_{b, tb}) = \mathbf{0.31 \text{ kNm/m}}$$

Total torque (ULS)

$$T_d = \max(T_{d, w}, T_{d, w, fav1}, T_{d, w, fav2}) \times L = \mathbf{1.55 \text{ kNm}}$$

Applied torque (SLS)

$$T_{d, w, ser} = \text{abs}(W_{sb, ser} \times e_{m, sb} + W_{tb, ser} \times e_{m, tb} + W_{sw, sb, ser} \times e_{b, sb} + W_{sw, tb, ser} \times e_{b, tb}) = \mathbf{0.23 \text{ kNm/m}}$$

Total torque (SLS)

$$T_{d, ser} = T_{d, w, ser} \times L = \mathbf{0.98 \text{ kNm}}$$

STEEL BEAM TORSION DESIGN (EN1993)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

Teds calculation version 1.0.04

Partial factors - Section 6.1

Resistance of cross-sections

$$\gamma_{M0} = \mathbf{1}$$

Resistance of members to instability

$$\gamma_{M1} = \mathbf{1}$$

Section details

Section type

UB 203x133x30 (BS4-1)

Steel grade - EN 10025-2:2004

S275

Nominal thickness of element

$$t_{nom} = \max(t_f, t_w) = \mathbf{9.6 \text{ mm}}$$

Nominal yield strength

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

Nominal ultimate tensile strength

$$f_u = \mathbf{410 \text{ N/mm}^2}$$

Modulus of elasticity

$$E = \mathbf{210000 \text{ N/mm}^2}$$

Shear centre

Distance between flange shear centres

$$h_s = h - t_f = \mathbf{197.2 \text{ mm}}$$

Shear centre (above bottom flange centroid)

$$e_{s, bf} = h_s / 2 = \mathbf{98.6 \text{ mm}}$$

Analysis results

Design bending moment - major axis

$$M_{y, Ed} = \mathbf{18.2 \text{ kNm}}$$

Design shear force - major axis

$$V_{y, Ed} = \mathbf{17.3 \text{ kN}}$$

Classification**Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)**

Width of section

$$c = d = \mathbf{172.4 \text{ mm}}$$

$$c / t_w = 26.9 = 29.1 \times \epsilon \leq 72 \times \epsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = \mathbf{56.2 \text{ mm}}$$

$$c / t_f = 5.8 = 6.3 \times \epsilon \leq 9 \times \epsilon \quad \text{Class 1}$$

Section is class 1

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Extension and Alterations

Initials

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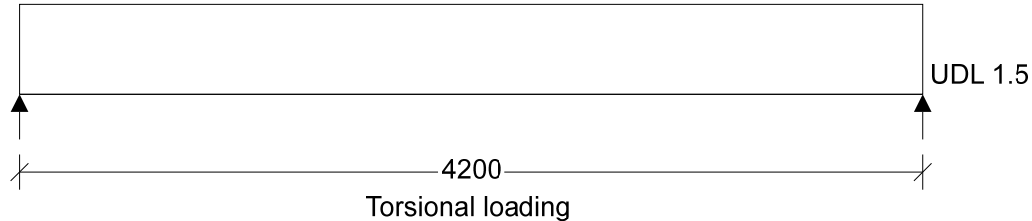
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Torsional loads



Load No.	Load type	Load (kNm)	Distance along beam (mm)
1	UDL	1.5	-

Rotation ϕ (SCI P385 Appendix C Case 4)

$$\phi = T_d \times a^2 / (G_{SEC3} \times I_t \times L) \times ((x \times L - x^2) / (2 \times a^2) + \cosh(x/a) - \tanh(L/(2 \times a)) \times \sinh(x/a) - 1)$$

Additional minor moment due to rotation, $M_{z,add,Ed} = M_{y,Ed} \times \phi$ Additional minor design moment due to rotation $M_{z,add,Ed} = 1.19 \text{ kNm}$ St Venant ϕ' (SCI P385 Appendix C Case 4)

$$\phi' = T_d \times a / (G_{SEC3} \times I_t \times L) \times (L / (2 \times a) - x/a + \sinh(x/a) - \tanh(L/(2 \times a)) \times \cosh(x/a))$$

Design value of the internal St Venant torsion moment $T_{t,Ed} = G_{SEC3} \times I_t \times \phi'$ St Venant torsion design moment $T_{t,Ed} = 0.43 \text{ kNm}$ Warping ϕ'' (SCI P385 Appendix C Case 4)

$$\phi'' = T_d / (G_{SEC3} \times I_t \times L) \times (-1 + \cosh(x/a) - \tanh(L/(2 \times a)) \times \sinh(x/a))$$

Warping design moment, $M_{w,Ed} = E_{SEC3} \times I_w \times \phi'' / h_s$ Warping design moment $M_{w,Ed} = 1.36 \text{ kNm}$ Warping torsional ϕ''' (SCI P385 Appendix C Case 4)

$$\phi''' = T_d / (G_{SEC3} \times I_t \times L \times a) \times (\sinh(x/a) - \tanh[L/(2 \times a)] \times \cosh(x/a))$$

Warping torsional design moment, $T_{w,Ed} = E_{SEC3} \times I_w \times \phi'''$ Warping torsional design moment $T_{w,Ed} = 0.35 \text{ kNm}$

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = 187.6 \text{ mm}$$

$$\eta = 1.000$$

$$h_w / t_w = 29.3 = 31.7 \times \epsilon / \eta < 72 \times \epsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{y,Ed} = 17.33 \text{ kN}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 1458 \text{ mm}^2$$

Design shear resistance - cl 6.2.6(2)

$$V_{pl,y,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 231.4 \text{ kN}$$

Shear stress due to St Venant torsion

$$\tau_{t,Ed} = T_{t,Ed} \times t_w / I_t = 26.43 \text{ N/mm}^2$$

Reduced shear resistance due to torsion - eq 6.26

$$V_{c,y,Rd} = V_{pl,T,y,Rd} = \sqrt{(1 - \tau_{t,Ed} / (1.25 \times (f_y / \sqrt{3}) / \gamma_{M0}))} \times V_{pl,y,Rd} = 215.5 \text{ kN}$$

$$V_{y,Ed} / V_{pl,T,y,Rd} = 0.08$$

PASS - Design shear resistance exceeds design shear force

Check bending moment - Section 6.2.5

Design bending moment

$$M_{y,Ed} = 18.2 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,y,Rd} = M_{pl,y,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 86.5 \text{ kNm}$$

$$M_{y,Ed} / M_{pl,y,Rd} = 0.21$$

PASS - Design bending resistance moment exceeds design bending moment

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Slenderness ratio for lateral torsional buckling

Loading factor C_1

$$C_1 = 1.127$$

Loading factor C_2

$$C_2 = 0.454$$

Loading factor C_3

$$C_3 = 0.52$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$$

Unrestrained effective length

$$L = 4200 \text{ mm}$$

Distance from shear centre to level of load

$$z_g = (h_{tb} / 2 \times w_{tb} - h_{tb} / 2 \times w_{sb}) / (w_{tb} + w_{sb}) = 45.2 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (k_{z,cr} \times L^2) \times (\sqrt{((k_{z,cr} / k_{w,cr})^2 \times I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z) + (C_2 \times z_g)^2) - (C_2 \times z_g)}) = 75.6 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 1.069$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

Lateral torsional buckling cannot be ignored

Check buckling resistance - Section 6.3.2.1

Buckling curve - Table 6.5

b

Imperfection factor - Table 6.3

$$\alpha_{LT} = 0.34$$

Correction factor for rolled sections

$$\beta = 0.75$$

LTB reduction determination factor

$$\Phi_{LT} = 0.5 \times (1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2) = 1.042$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / (\Phi_{LT} + \sqrt{(\Phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}), 1, 1 / \bar{\lambda}_{LT}^2) = 0.657$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.975$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1, 1 / \bar{\lambda}_{LT}^2) = 0.674$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 58.3 \text{ kNm}$$

$$M_{y,Ed} / M_{b,y,Rd} = 0.312$$

PASS - Design buckling resistance exceeds design buckling moment

Check bending moment - Section 6.2.5

Design bending moment

$$M_{z,Ed,total} = M_{z,Ed} + M_{z,add,Ed} = 1.2 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,z,Rd} = M_{pl,z,Rd} = W_{pl,z} \times f_y / \gamma_{M0} = 24.3 \text{ kNm}$$

$$M_{z,Ed,total} / M_{pl,z,Rd} = 0.049$$

PASS - Design bending resistance moment exceeds design bending moment

Check warping moment

Warping moment in flange

$$M_{w,Ed} = 1.36 \text{ kNm}$$

Plastic modulus of flange

$$W_{pl,f} = t_f \times b^2 / 4 = 43.03 \text{ cm}^3$$

Design warping resistance of flange

$$M_{w,Rd} = W_{pl,f} \times f_y / \gamma_{M0} = 11.83 \text{ kNm}$$

$$M_{w,Ed} / M_{w,Rd} = 0.115$$

PASS - Bending resistance in one flange exceeds the design warping moment

Combined bending and torsion (EN1993-6 Annex A)

Equiv. uniform moment factor (parabolic curve)

$$C_{mz} = 0.95$$

Characteristic moment resistance - y axis

$$M_{y,Rk} = W_{pl,y} \times f_y = 86.5 \text{ kNm}$$

Characteristic moment resistance - z axis

$$M_{z,Rk} = W_{pl,z} \times f_y = 24.3 \text{ kNm}$$

Characteristic warping resistance

$$M_{w,Rk} = W_{pl,f} \times f_y = 11.8 \text{ kNm}$$

Interaction factors

$$k_w = 0.7 - 0.2 \times M_{w,Ed} / (M_{w,Rk} / \gamma_{M1}) = 0.68$$

$$k_{zw} = 1 - M_{z,Ed,total} / (M_{z,Rk} / \gamma_{M1}) = 0.95$$

$$k_\alpha = 1 / (1 - M_{y,Ed} / M_{cr}) = 1.32$$

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Interaction formula eqn A.1	$M_{y,Ed} / (\chi_{LT} \times M_{y,Rk} / \gamma_{M1}) + C_{mz} \times M_{z,Ed,total} / (M_{z,Rk} / \gamma_{M1}) + k_w \times k_{zw} \times k_{\alpha} \times M_{w,Ed} / (M_{w,Rk} / \gamma_{M1}) = 0.464$ PASS - Combined bending and torsion check satisfied
Plastic verification - Exp.6.41	$(M_{y,Ed} / M_{pl,y,Rd})^2 + M_{z,Ed,total} / M_{pl,z,Rd} + M_{w,Ed} / M_{w,Rd} = 0.208$ PASS - Plastic interaction criterion is less than 1.0
Serviceability limit checks	
Rotation limit	$\phi_{ser,lim} = 3.00 \text{ deg}$
Rotation of torsion beam	$\phi_{ser} = M_{z,add,Ed} \times 180 / (M_{y,Ed} \times \gamma_G \times \pi) = 2.77 \text{ deg}$ PASS - Rotation limit exceeds rotation in torsion beam
Vertical deflection limit	$\delta_{v,lim} = 10.0 \text{ mm}$
SLS loading on beam	$f_{d,ser} = W_{sb,ser} + W_{tb,ser} + W_{sw,sb,ser} + W_{sw,tb,ser} = 5.93 \text{ kN/m}$
Vertical deflection of torsion beam	$\delta_v = 5 \times f_{d,ser} \times L^4 / (384 \times E_{sb} \times I_{y,b}) = 4.0 \text{ mm}$ PASS - Vertical deflection limit exceeds vertical deflection in torsion beam

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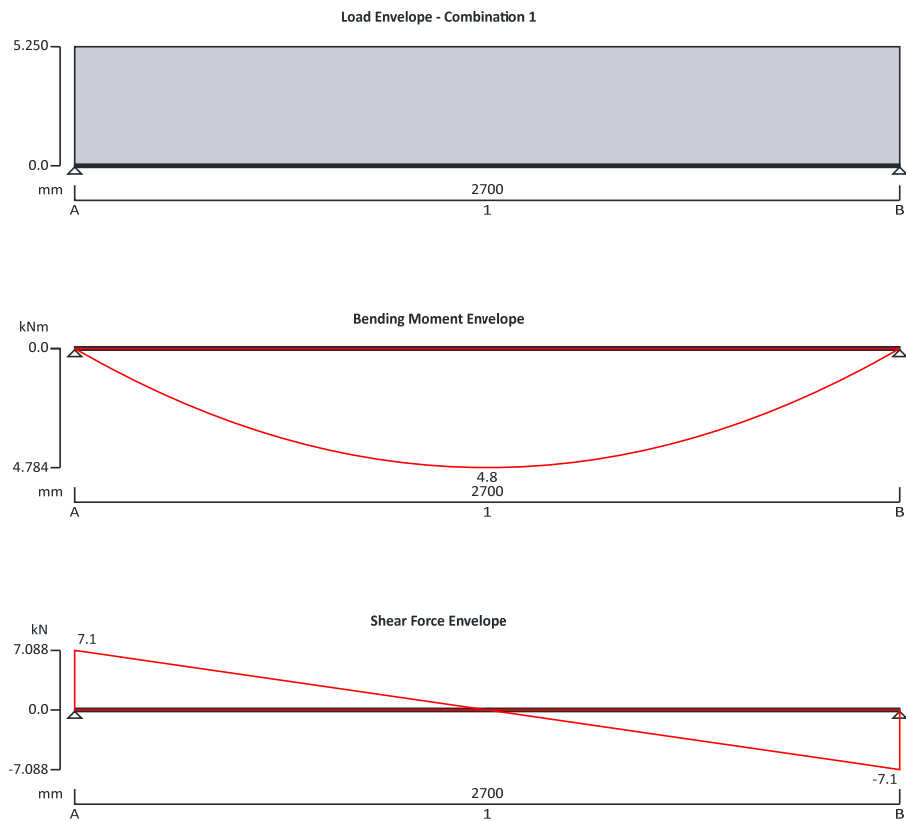
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POST P1

STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14



Support conditions

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

Applied loading

Beam loads	UDL wk - Variable full UDL 3.5 kN/m
------------	-------------------------------------

Load combinations

Load combination 1	Support A	Permanent $\times$ 1.35
		Variable $\times$ 1.50
	Support B	Permanent $\times$ 1.35
		Variable $\times$ 1.50

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Analysis results

Maximum moment

$M_{\max} = 4.8 \text{ kNm}$

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

Maximum shear

$V_{\max} = 7.1 \text{ kN}$

$V_{\min} = -7.1 \text{ kN}$

Deflection

$\delta_{\max} = 4.1 \text{ mm}$

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

Maximum reaction at support A

$R_{A_{\max}} = 7.1 \text{ kN}$

$R_{A_{\min}} = 7.1 \text{ kN}$

Unfactored variable load reaction at support A

$R_{A_{\text{Variable}}} = 4.7 \text{ kN}$

Maximum reaction at support B

$R_{B_{\max}} = 7.1 \text{ kN}$

$R_{B_{\min}} = 7.1 \text{ kN}$

Unfactored variable load reaction at support B

$R_{B_{\text{Variable}}} = 4.7 \text{ kN}$

Section details

Section type

SHS 100x100x5.0 (Tata Steel Celsius (Gr355 Gr420 Gr460))

Steel grade

S355H**EN 10210-1:2006 - Hot finished structural hollow sections of non-alloy and fine grain steels**

Nominal thickness of element

$t = 5.0 \text{ mm}$

Nominal yield strength

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

Nominal ultimate tensile strength

$f_u = 470 \text{ N/mm}^2$

Modulus of elasticity

$E = 210000 \text{ N/mm}^2$

Partial factors - Section 6.1

Resistance of cross-sections

$\gamma_{M0} = 1.00$

Resistance of members to instability

$\gamma_{M1} = 1.00$

Resistance of tensile members to fracture

$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis

$K_y = 1.000$

Effective length factor in minor axis

$K_z = 1.000$

Effective length factor for torsion

$K_{LT,A} = 1.000$

$K_{LT,B} = 1.000$

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{235 \text{ N/mm}^2 / f_y} = 0.81$

Internal compression parts subject to bending and compression - Table 5.2 (sheet 1 of 3)

Width of section

$c = h - 3 \times t = 85 \text{ mm}$

$\alpha = \min([h / 2 + N_{Ed} / (2 \times t \times f_y) - t] / c, 1) = 0.587$

$c / t = 20.9 \times \varepsilon \leq 396 \times \varepsilon / (13 \times \alpha - 1) \quad \text{Class 1}$

Internal compression parts subject to compression only - Table 5.2 (sheet 1 of 3)

Width of section

$c = b - 3 \times t = 85 \text{ mm}$

$c / t = 20.9 \times \varepsilon \leq 33 \times \varepsilon$

Class 1

Section is class 1**Check shear - Section 6.2.6**

Height of web

$h_w = h - 2 \times t = 90 \text{ mm}$

Shear area factor

$\eta = 1.000$

$h_w / t < 72 \times \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force

$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 7.1 \text{ kN}$

Shear area - cl 6.2.6(3)

$A_v = A \times h / (b + h) = 937 \text{ mm}^2$

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Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 192 \text{ kN}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 4.8 \text{ kNm}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 23.6 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = 0.94$$

$$C_1 = 1 / k_c^2 = 1.132$$

Curvature factor

$$g = 1$$

Poissons ratio

$$\nu = 0.3$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$$

Unrestrained length

$$L = 1.0 \times L_{s1} = 2700 \text{ mm}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 600.9 \text{ kNm}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.198$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = 0.4$$

$\bar{\lambda}_{LT} < \bar{\lambda}_{LT,0}$ - Lateral torsional buckling can be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5

$$d$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = 0.76$$

Correction factor for rolled sections

$$\beta = 0.75$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.438$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 1.000$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.992$$

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 1.000$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 23.6 \text{ kNm}$$

PASS - Design buckling resistance moment exceeds design bending moment

Check compression - Section 6.2.4

Design compression force

$$N_{Ed} = 17.3 \text{ kN}$$

Design resistance of section - eq 6.10

$$N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0} = 665 \text{ kN}$$

Slenderness ratio for major (y-y) axis buckling

Critical buckling length

$$L_{cr,y} = L_{s1} \times K_y = 2700 \text{ mm}$$

Critical buckling force

$$N_{cr,y} = \pi^2 \times E_{SEC3} \times I_y / L_{cr,y}^2 = 794.5 \text{ kN}$$

Slenderness ratio for buckling - eq 6.50

$$\bar{\lambda}_y = \sqrt{[A \times f_y / N_{cr,y}]} = 0.915$$

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2

$$a$$

Imperfection factor - Table 6.1

$$\alpha_y = 0.21$$

Buckling reduction determination factor

$$\phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2] = 0.994$$

Buckling reduction factor - eq 6.49

$$\chi_y = \min(1 / [\phi_y + \sqrt{(\phi_y^2 - \bar{\lambda}_y^2)}], 1) = 0.724$$

Design buckling resistance - eq 6.47

$$N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1} = 481.5 \text{ kN}$$

PASS - Design buckling resistance exceeds design compression force

Slenderness ratio for minor (z-z) axis buckling

Critical buckling length

$$L_{cr,z} = L_{s1_seg1} \times K_z = 2700 \text{ mm}$$

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Critical buckling force

$$N_{cr,z} = \pi^2 \times E_{SEC3} \times I_z / L_{cr,z}^2 = \mathbf{794.5 \text{ kN}}$$

Slenderness ratio for buckling - eq 6.50

$$\bar{\lambda}_z = \sqrt{[A \times f_y / N_{cr,z}]} = \mathbf{0.915}$$

Design resistance for buckling - Section 6.3.1.1

Buckling curve - Table 6.2

a

Imperfection factor - Table 6.1

$$\alpha_z = \mathbf{0.21}$$

Buckling reduction determination factor

$$\phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2] = \mathbf{0.994}$$

Buckling reduction factor - eq 6.49

$$\chi_z = \min(1 / [\phi_z + \sqrt{(\phi_z^2 - \bar{\lambda}_z^2)}], 1) = \mathbf{0.724}$$

Design buckling resistance - eq 6.47

$$N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1} = \mathbf{481.5 \text{ kN}}$$

PASS - Design buckling resistance exceeds design compression force**Combined bending and axial force - Section 6.2.9**

Bending and axial force check - eq 6.33 & 6.34

$$N_{Ed} \leq \min(0.25 \times N_{pl,Rd}, 0.5 \times h_w \times t \times f_y / \gamma_{M0})$$

No allowance on the plastic moment need to be accounted for due to the effect of axial force**Check combined bending and compression - Section 6.3.3**

Equivalent uniform moment factors - Table B.3

$$M_{hy} = \mathbf{0 \text{ kNm}}$$

$$M_{sy} = \mathbf{5 \text{ kNm}}$$

$$\psi_y = \mathbf{1.000}$$

$$\alpha_{hy} = M_{hy} / M_{sy} = \mathbf{0.000}$$

$$C_{my} = 0.95 + 0.05 \times \alpha_{hy} = \mathbf{0.950}$$

$$M_{hz} = \mathbf{0 \text{ kNm}}$$

$$M_{sz} = \mathbf{0 \text{ kNm}}$$

$$\psi_z = \mathbf{1.000}$$

$$C_{mz} = 0.6 + 0.4 \times \psi_z = \mathbf{1.000}$$

$$M_{hLT} = \mathbf{0 \text{ kNm}}$$

$$M_{sLT} = \mathbf{5 \text{ kNm}}$$

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

$$\alpha_{hLT} = M_{hLT} / M_{sLT} = \mathbf{0.000}$$

$$C_{mLT} = 0.95 + 0.05 \times \alpha_{hLT} = \mathbf{0.950}$$

Interaction factors k_{ij} for members not susceptible to torsional deformations - Table B.1

Characteristic moment resistance

$$M_{Rk} = W_{pl,y} \times f_y = \mathbf{23.6 \text{ kNm}}$$

Characteristic resistance to normal force

$$N_{Rk} = A \times f_y = \mathbf{665 \text{ kN}}$$

Interaction factors

$$k_{yy} = C_{my} \times [1 + \min(\bar{\lambda}_y - 0.2, 0.8) \times N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1})] = \mathbf{0.974}$$

$$k_{zy} = 0.6 \times k_{yy} = \mathbf{0.585}$$

Interaction formulae - eq 6.61 & eq 6.62

$$N_{Ed} / (\chi_y \times N_{Rk} / \gamma_{M1}) + k_{yy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1}) = \mathbf{0.234}$$

$$N_{Ed} / (\chi_z \times N_{Rk} / \gamma_{M1}) + k_{zy} \times M_{Ed} / (\chi_{LT} \times M_{Rk} / \gamma_{M1}) = \mathbf{0.155}$$

PASS - Combined bending and compression checks are satisfied**Check vertical deflection - Section 7.2.1**

Consider deflection due to permanent and variable loads

Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{7.5 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{4.127 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

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Support @ A.Vertical load from FB2/FB3 - $N_{Ed} = 55.3 + 32.2 = 87.5 \text{ kN}$.Capacity $N_{Rd} = \beta b t f_c / \gamma_m$ Alf.

$$\beta = \frac{h_{eff}}{E_{eff}} = \frac{2400}{230} = 10.4, \quad e = 0.1t \quad \beta = 0.86$$

$$b = 330 \text{ mm}$$

$$t = 250 \text{ mm}$$

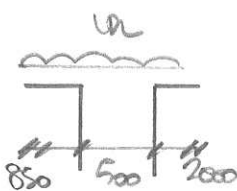
$$f_c = 5.0 \text{ N/mm}^2 \quad \gamma_m = 3.1$$

$$\text{Alf} = 0.7 + 1.5A = 0.82$$

$$\therefore N_{Rd} = 0.86 \times 330 \times 250 \times 5.0 \times 0.82$$

$$= \underline{96.0 \text{ kN}} \quad \text{OK}$$

∴ provide 330x250 solid brick pier.

Support @ B.

$$\text{VEd} = g_k = 1.8 \text{ kN/m} \\ q_k = 3.6 \text{ kN/m} \quad \left. \begin{array}{l} \\ \end{array} \right\} N_{Ed} = 7.0 \text{ kN/m}$$

$$\text{Force in pier } N_{Ed} = 7.0 \left(\frac{2.0 + 0.05}{2} + 0.5 \right) = 15.0 \text{ kN}$$

$$\beta = \frac{h_{eff}}{E_{eff}} = \frac{2400}{100} = 24 \quad e = 0.05t \quad \beta = 0.53$$

$$b = 500 \text{ mm} \quad t = 100 \text{ mm}$$

$$f_c = 3.5 \text{ N/mm}^2 \quad \gamma_m = 3.1$$

$$\text{Alf} = 0.78$$

$$\therefore N_{Rd} = 0.53 \times 500 \times 100 \times 3.5 \times 0.78$$

$$= \underline{23.4 \text{ kN}} \quad \text{OK}$$

∴ Retained pier is satisfactory.